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Instream Aeration Of Polluted Rivers

WILLIAM WHIPPLE, JR.
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WATER RESOURCES RESEARCH INSTITUTE
RUTGERS UNIVERSITY
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NEW BRUNSWICK, NEW JERSEY 08903
AUGUST, 1969

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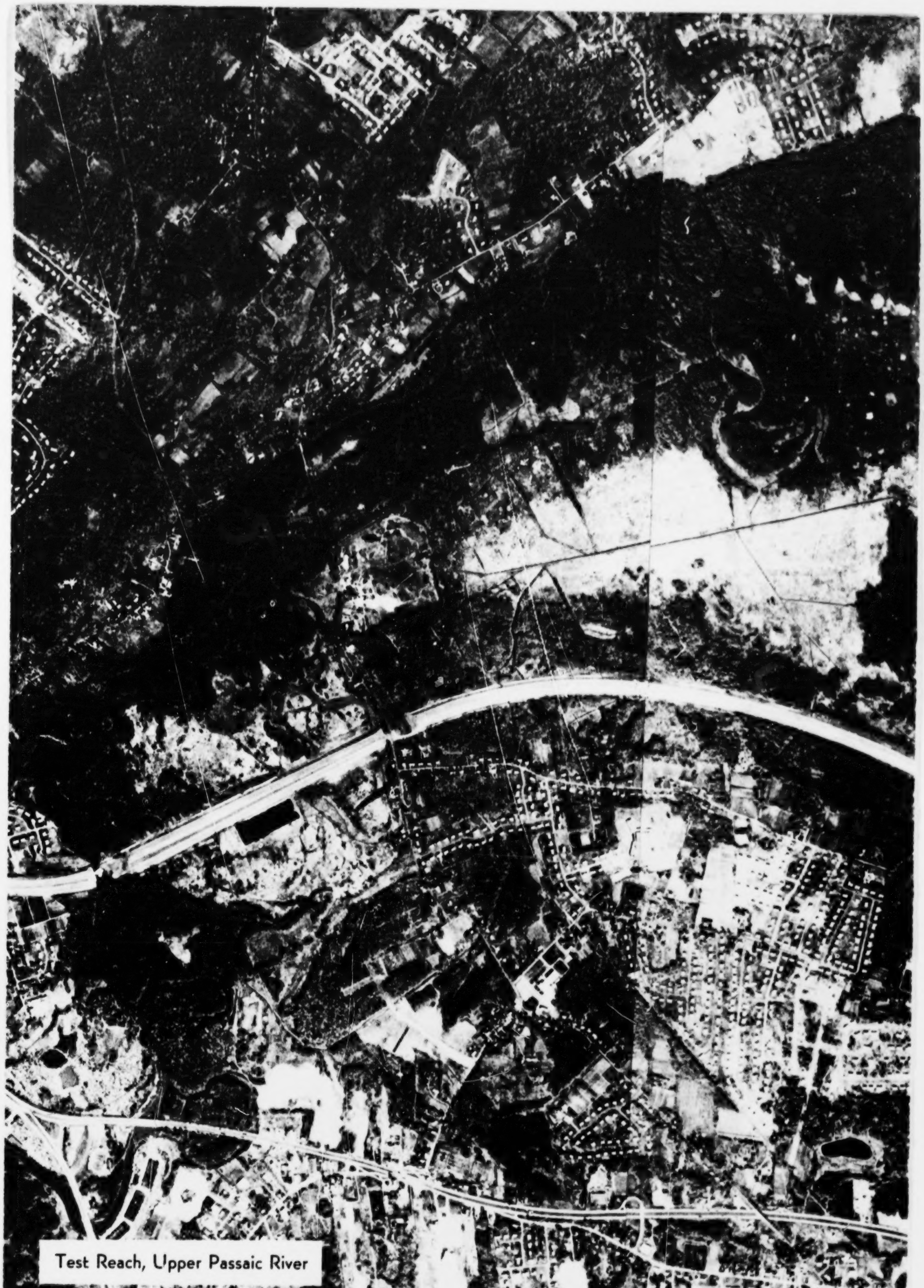

Test Reach, Upper Passaic River

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PREFACE

Our truly impressive national system of water quality standards was established rapidly, with widespread public support, without waiting for quantitative supporting studies. The environmental goals thus established are unquestionably desirable, but it is still not clear just how such goals are to be achieved, how much the program will cost, and how the program is to be enforced. Preliminary indications are that the program will be much more difficult and expensive than had been anticipated.

Unfortunately, while several generations have worked on the problems of processing waste in treatment plants, relatively little attention has been given to the processes which occur or which might be made to occur in the river itself. The assumption has too often been made that the recorded effluents coming from treatment plants constitute the problem, and that all that has to be done is to improve efficiency of treatment in order to achieve any degree of water quality desired. Unfortunately, research is now beginning to show that, even in reasonably well administered areas, the recorded effluents may be only one half, or perhaps one third, of the total pollution loading entering the river system. In some cases it may prove impracticable to achieve the enacted water quality standards by any degree of treatment.

Almost four years ago, a group of engineers and scientists, mostly at Rutgers University, banded together to investigate this problem in depth, and, in particular, to determine the potentialities of instream river aeration as a supplemental means of water quality control. The idea was not original. In fact aeration devices have been tried out in rivers in a number of cases, starting well over 40 years ago, and within the last year renewed interest has been shown in such possibilities. What was original was to combine four essential aspects. First, the tests were made with full-scale commercially-available equipment, professionally engineered and costed. Second, a test reach was located on a small polluted river extending 12 miles without a tributary or additional source of pollution, constituting, in effect, a long biochemical reactor. Third, all aspects of the process were subject to scientific investigation in connection with related research projects. Fourth, modern techniques of basin-wide systems analysis were employed to compare performance of aerator systems to alternatives of improved waste treatment.

The results of the analysis provide definite criteria of cost and of effectiveness of instream aeration systems, and establish a sound basis showing that such aeration constitutes an economical means of maintaining desired dissolved oxygen levels in streams. The presumption is strong that on any river where secondary treatment of effluents is insufficient to achieve desired dissolved oxygen levels, instream aeration will be more economical than advanced waste treatment. Possibilities of low flow augmentation should be compared with costs of instream aeration in the computation of benefit.

From a scientific viewpoint, the tests have disclosed a rather surprising phenomenon. The stream below the aerator is subject to a deoxygenation rate several times greater than exists in the stream in its undisturbed state, and this effect extends for several miles downstream. The hypothesis that this increase indicates the triggering of nitrogenous biochemical oxygen demand requires further research, but if verified would constitute an additional advantage of instream aeration.

Assuming that the economic and technological conclusions of this report are found generally acceptable, some serious problems must be faced before induced aeration can realize its potential value. In the first place, our present institutions have established a rather awkward institutional balance in water quality control. Construction and maintenance of treatment plants are decentralized to municipalities and industries, enforcement is largely in the hands of states, and policy and planning are largely Federal. Induced river aeration seems best adaptable to a river basin organization, which exists in only a few cases. Some one must make sure that the aerators do not furnish a pretext for postponing needed waste treatment measures. The planning for such aerators will require a considerable degree of scientific expertise. Moreover, who should pay for the aerators?

Such problems can be troublesome, no doubt, but the relative advantage of the aerators appears sufficient to warrant facing them. So far as this report is concerned, a technological and economic solution is offered, which has the potential of avoiding a vast waste of resources upon less productive alternatives, if our legally established water quality standards are indeed to be achieved.

Princeton, N. J.
July 6, 1969

William Whipple, Jr.

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THE WATER RESOURCES RESEARCH INSTITUTE

The Water Resources Research Institute of Rutgers University - The State University of New Jersey was organized for the State of New Jersey under authority of P.L. 88-379. Although primarily engaged at Rutgers University, its program currently includes research at both Princeton University and Stevens Institute. The activities of the Institute are controlled by the Water Resources Research Council, the members of which are the following:

Dr. Elmer C. Easton, Dean, College of Engineering, Rutgers, Chairman of Council, 1968-69.

Dr. Marvin L. Granstrom, Chairman, Department of Civil Engineering, Rutgers.

Dr. Harold H. Haskin, Professor, Department of Zoology, Rutgers.

Dr. Joseph V. Hunter, Associate Research Specialist, Department of Environmental Science, Rutgers.

Dr. Roscoe P. Kandle, Commissioner, State Department of Health, Trenton, New Jersey.

Dr. Leland C. Merrill, Jr., Dean, College of Agriculture and Environmental Science, Rutgers, Chairman of Council, 1969-70.

Mr. Robert A. Roe, Commissioner, State Department of Conservation and Economic Development, Trenton, New Jersey.

Dr. Harry F. Stark, Ass't Dean, University Extension Division, Rutgers.

Brig. Gen. William Whipple, Jr., (ret'd), Director, Water Resources Research Institute, Rutgers.

Chapter I

PROGRAM OBJECTIVES AND THE TEST RIVER

Objectives and Scope of Program

Increasingly it has come to be realized that the classic pattern of alleviating stream pollution by treatment of individual waste sources, occasionally supplemented by low flow augmentation, is inadequate in the light of modern technology. For the large proportion of water quality situations involving organic pollution and deficiencies in dissolved oxygen, an alternative approach is available, namely the artificial re-aeration of the stream itself, or, as it may be expressed, induced aeration.* This study results from a program of demonstration and research in induced aeration of rivers. Field work with full scale equipment was conducted on a small polluted river, the Passaic River in New Jersey.

For the proper evaluation of aerator performance, the effects were studied not only in the immediate vicinity but downstream for a considerable distance. Along with aerator efficiency, special studies included algal photosynthesis and respiration, benthal oxygen demand, first and second stage biochemical oxygen demand, and observations of turbulence downstream of the aerator. These studies provided coefficients for dynamic modelling of the dissolved oxygen profile. It is the total effect of the aerators on the river downstream which determines their value, either singly or as a part of a system.

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*Such aeration is applicable to dissolved oxygen deficiency, but not, of course, to problems of pathogenics, nutrients or non-degradables. See references (1), (2), (3), (4).

The two basic types of aeration devices were tested, namely, surface mechanical aerators and diffusion (bubble) aerators.* Both types are widely used in waste treatment processes. However, the efficiency or oxygen transfer rate of aeration equipment in flowing streams is considerably lower than the rates applicable to non-flow tanks and lagoons, and varies markedly with stream flow conditions. This point is of basic importance to what is to follow. The manufacturers' guarantee rate of 4.0 pounds of oxygen per horsepower hour, or more, applies to "standard conditions," which includes water at zero dissolved oxygen and 20°C.** However, in rivers, oxygen uptake may fall to 1.0 pound per horsepower hour of installed capacity, or even lower. Part of the difference is due to the higher initial dissolved oxygen level, and to ground rules of the manufacturer, such as using shaft horsepower rather than motor horsepower, but another important factor seems to stem from the continual recirculation of water when aerators are used in slow moving streams. See Figure 1-1. The recirculation causes the water in the aeration zone to approximate the dissolved oxygen level downstream rather than that of the unaerated water. If the water recirculated actually averaged 4 mg/l dissolved oxygen rather than 2.0 mg/l of water from upstream, at a temperature of 26°C, the oxygen transfer rate would be reduced by one third below that anticipated, by this reason alone.*** If the objective is to maintain

---

*See Chapter II for description of equipment.

**Kalinske (5) states that surface aerators (in tanks and lagoons) can supply oxygen at 2.8-4.5 lbs per hp-hr at standard conditions. Linsley and Franzini (16) state that aerators in rivers commonly transfer 2 to 4 lbs of oxygen per hp-hr.

***Oxygen transfer varies as the oxygen deficit or "driving force" (C_s - C). For detailed discussion, refer to Chapter III.

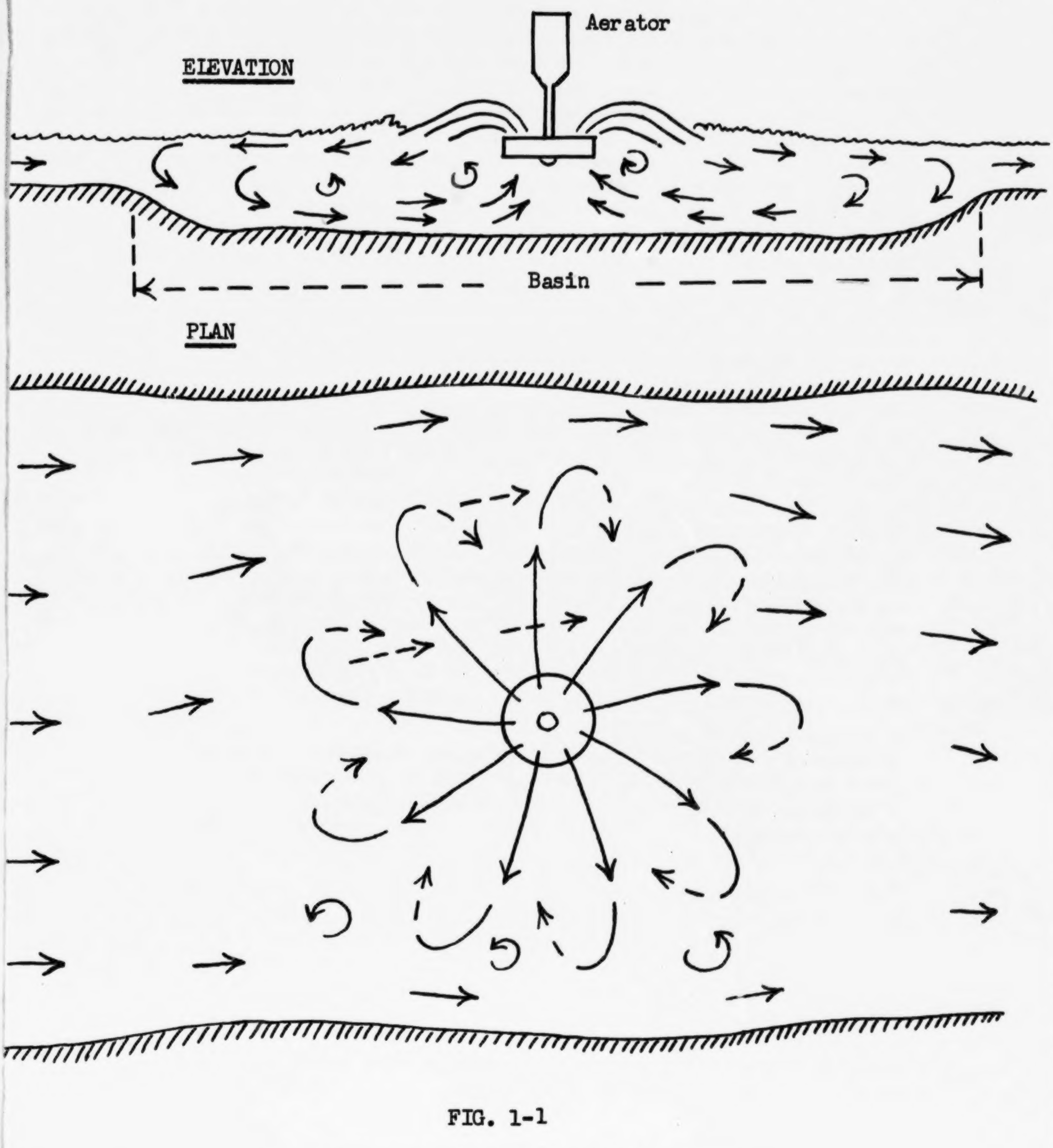
Aerator
ELEVATION
Basin
PLAN

FIG. 1-1

WATER CIRCULATION OF AERATOR

a minimum standard of 4 or 5 mg/l, the difference will be even more dramatic. This effect is discussed in more detail in Chapter II.

Also, it has been found that the use of aerators in rivers may have the effect of greatly increasing the deoxygenation coefficient. The dynamics of biochemical oxygen demand of the river must be re-modelled to take account of such complications. Even now, certain questions remain unresolved, leaving a margin of uncertainty as to the value of certain parameters, and a necessity for continuation of research. However, it is abundantly clear that oxygen transfer rates derived from treatment plant experience are not directly applicable to dissolved oxygen deficiencies in rivers, for several reasons. The present report provides a basis for estimates of aerator effectiveness when used in rivers. The economic analysis of this report indicates that induced aeration will probably be useful in many rivers, in situations such that secondary treatment of effluents is not sufficient to achieve desired dissolved oxygen standards. For such rivers, induced aeration is an alternative to low flow augmentation by reservoir storage, or advanced waste treatment.

This program grew out of a demonstration project, "Oxygen Regeneration of Polluted Rivers," supported by the Federal Water Pollution Control Administration and the Department of Conservation and Economic Development, State of New Jersey.* The present report also includes results from four related research projects funded by the Office of Water Resources Research, Department of the Interior, with matching funds from the N. J. Department of Conservation and Economic Development** In addition to participating faculty members (all from Rutgers University except as indicated), consulting engineers Hazen and Sawyer were retained for selection of equipment, design and supervision of construction, and cost estimates. Cooperation and assistance were available from the Morris County Freeholders and Morris County Mosquito Extermination Commission, particularly Mr. Robert L. Vannote, who arranged for donation of rights of way, and carried out the necessary dredging of basins at cost. The New Jersey office of the U. S. Geological Survey, headed by Mr. John McCall, assisted by furnishing data and making special discharge readings and checks of the little used Pine Brook gage. During 1968, coordination was also arranged with the field offices of the Federal Water Pollution Control Administration, headed by Mr. Kenneth Walker. The analytical part of the work benefitted from the extensive water quality data obtained through the Passaic Valley Water Commission and made available through the courtesy of Mr. Wendell R. Imhoffer and Mr. Frank J. DeHooge, and from the official records as to treatment plants and effluents of the N. J. Department of Public Health, Mr. Ernest Segesser.

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*Project Director: W. Whipple; original principal investigators: J. V. Hunter, B. Davidson, Lucien Brush (Princeton University) and R. Peskin. Frank Dittman, Shaw Yu and George Mattingly (Princeton University) are now participating in the project.

**(1) "Photosynthetic Reaeration in the Upper Passaic River," J. V. Hunter (B-010-N.J.), (2) "Process Control Model for Oxygen Regeneration of Polluted Rivers," B. Davidson and J. V. Hunter (B-011-N.J.), (3) "Distributed and Multivariable System Theory," B. Davidson and V. J. Tarassov (A-017-N.J.), (4) "Urbanization and its Effect upon Water Resources," M. Marcus and W. Whipple (B-002-N.J.). In addition, there were several other related research projects, the results of which are not included in this report.

During the investigations the work was divided between those participating generally in accordance with research interests. Dr. Hunter was concerned mainly with photosynthesis, benthal effects, and water chemistry and biochemistry in general (Ch. III). Dr. Davidson was responsible for systems modelling and dynamic simulation, using analogue computer (Ch. VI). Dr. Dittman supervised field operations and made power studies (Ch. II). Dr. Yu participated in hydrology and evaluation of oxygen transfer rates (Ch. IV). General planning and coordination and the economic analysis was by W. Whipple (Ch. I, V, and VII). Dr. Brush and Dr. Peskin advised regarding dispersion and hydrodynamics.

The Passaic River

The eight counties of metropolitan Northern New Jersey, within which the Passaic River lies, have a combined population of well over four million persons. See Figure 1-2. The area in which pollution problems are important on the Passaic extends from Chatham 58 miles downstream to the estuary, and also on the lower Rockaway and

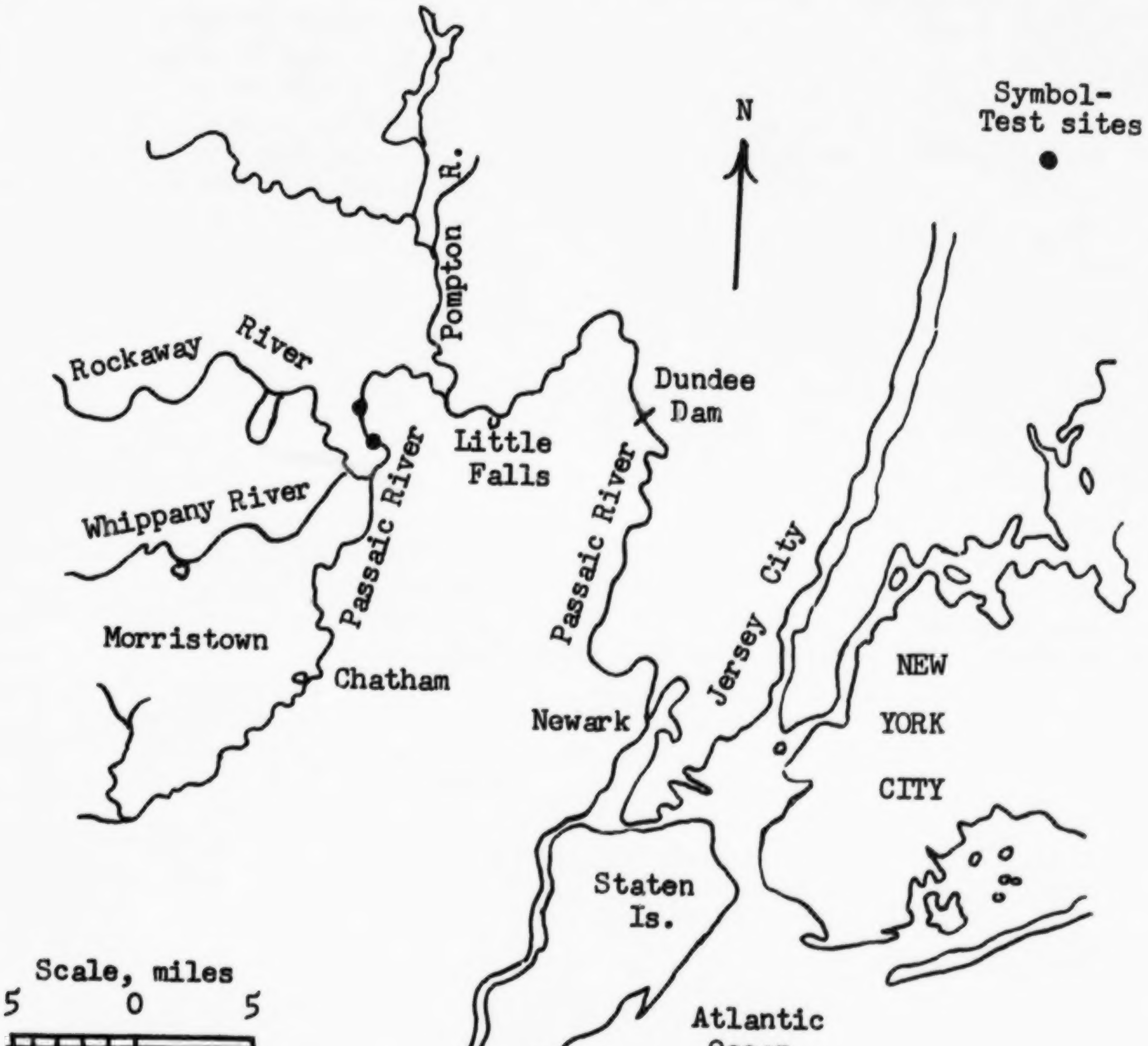
Symbol-
Test sites
●
N
Pompton R.
Rockaway River
Little Falls
Dundee Dam
Passaic River
Whippany River
Passaic River
Jersey City
Morristown
Chatham
NEW
YORK
CITY
Newark
Staten Is.
Scale, miles
5 0 5
Atlantic Ocean

FIG. 1-2

PASSAIC RIVER BASIN

Whippany Rivers. The most basic need is to maintain reasonable water quality above Little Falls, at which point there is a major diversion for municipal water supply.

The Passaic River above Little Falls is a winding sluggish stream, the channel of which has been extensively cleared, straightened, and dredged in the interest of expediting runoff and drainage, and aiding in control of mosquitoes. See frontispiece and Figure 1-3. In the reach Pine Brook - Two Bridges, within which the induced aeration operations were carried out, the mean stream velocity only exceeds one foot per second at discharges approximating 1,000 cfs. During low summer flows the discharge is usually around 100 - 125 cfs , the width is about one hundred feet, and the depth from two to four feet. Rhodomine B dye tests conducted by the U. S. Geological Survey indicate that under "medium low flow conditions" travel times of over 6 days might be expected for flows to move from Chatham to Little Falls, a distance of about 30 miles.(6) See Appendix I-1 to this chapter for further hydrologic detail.

Although the part of the basin above Little Falls has no large city, population is growing rapidly, being estimated at 239,500 in 1950 and 514,500 in 1966. This was a growth of 115%, or an average annual compounded growth rate of 4.9%,(8) as compared to the national average of 1.6%.(17) The very much larger populations in the lower part of the basin do not discharge much of their effluents into the river, and they affect the water situation mainly in their vast and ever-growing requirements for water supplies. Water is taken from the basin at a number of points at and above Little Falls. The mean annual flow at Little Falls, below the diversion, is 1147 cfs (1897-1966).

Studies of the Federal Water Pollution Control Administration in 1967 indicated that municipal and industrial waste load discharged to the Passaic River and its tributaries totaled 31,800 lbs/day of ultimate BOD.*(7)

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*That is, the pounds of oxygen which this quantity of waste will require for the completion of biochemical oxidation in the stream, over whatever period is required. Based mainly upon a 1962 survey.

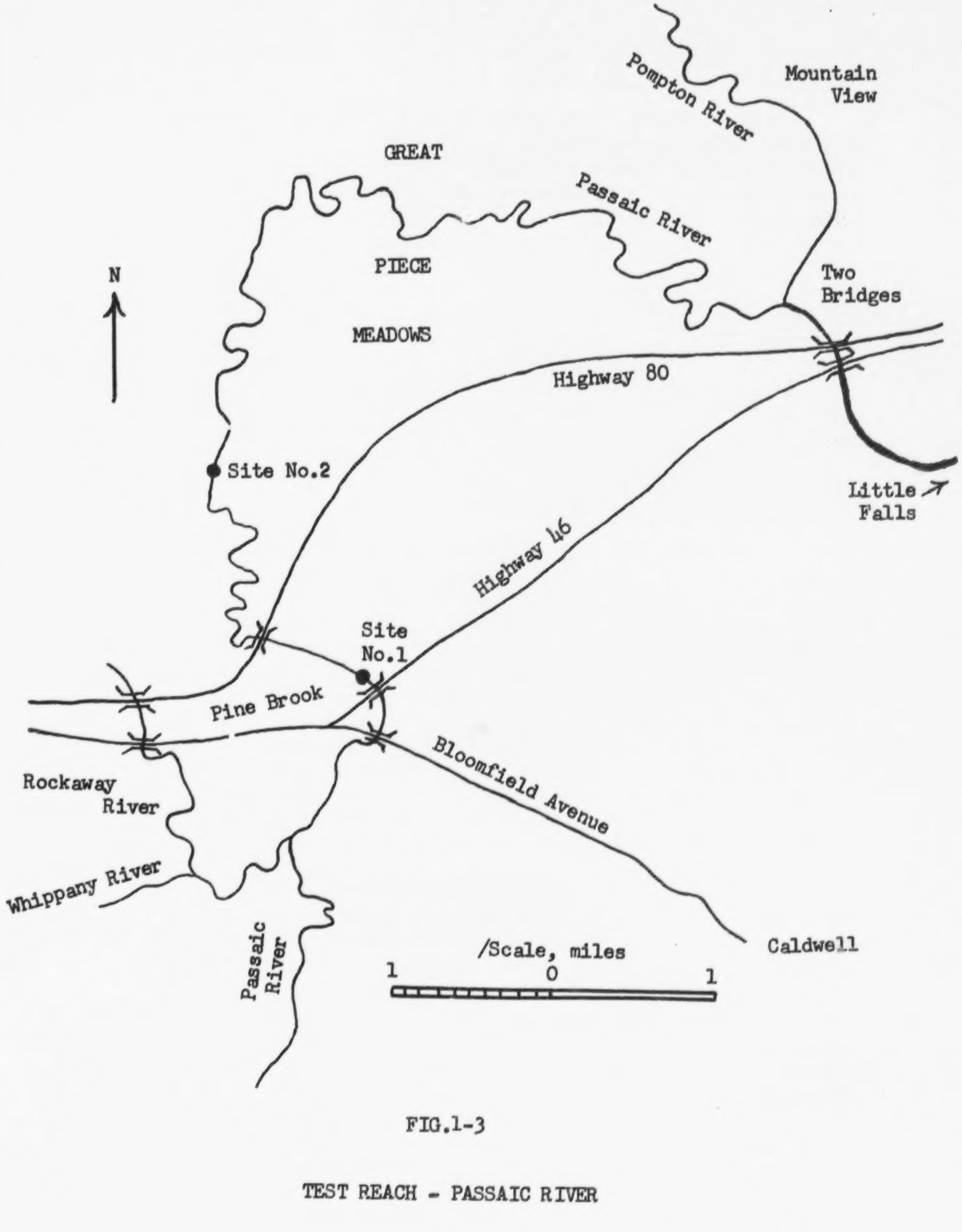
Mountain View
Pompton River
GREAT
PIECE
MEADOWS
Passaic River
Two Bridges
N
Highway 80
Site No.2
Little Falls
Highway 46
Site No.1
Pine Brook
Bloomfield Avenue
Rockaway River
Whippany River
Passaic River
Caldwell
/Scale, miles
1
0
1

FIG.1-3

TEST REACH - PASSAIC RIVER

A future organic loading of 47,200 lbs. of ultimate BOD per day has been forecast for the year 2020, or a 47.5% increase. This forecast is based upon an assumed continuation of present development trends and an increase in efficiency of waste treatment to an average of 90%. During the recent drought, from 1961-66, the level of pollution at and below the junction of the upper Raritan and Whippany Rivers (Figure 1-2) was so high as to cause anaerobic conditions over considerable periods of time. It was only the addition of the better quality Pompton River flows which made this water even minimally acceptable at Little Falls, and a few times it had to be rejected as impracticable to treat. Any further deterioration in water quality would very seriously affect the utility of this important water supply.

Organic Pollution Situation - Upper Passaic

Studies of the dynamics of biochemical oxygen demand in the critical upper part of the river have been made in connection with an associated research project.(8) These studies used the extensive data as to stream flow water quality obtained by the Passaic Valley Water Commission, and the records of the State Department of Public Health as to treatment plants. The results show that the total effluents discharged into the river above Twin Falls had about 27,500 lbs of BOD (ultimate) per day in 1967, which was not much changed over the preceding ten years.

The total organic loading entering the river system was very much higher than the total of the effluents. In order to determine quantities it was first necessary to evaluate applicable parameters. An analysis of BOD and dissolved oxygen covering a period of eleven years was made for the 12 miles reach between Pine Brook and Two Bridges (Figure 1-3). This reach is rather remarkable for a polluted river in that it includes neither any appreciable pollution source nor any tributary, and therefore is admirably suited for analysis. An average year-round value of K_r of 0.218 was found for the equation

$$\frac{dL}{dt} = -K_r L \qquad \text{.... Eq. 1-1}$$

where L is BOD (ult.), and t is time (days), in the usual first order equation for BOD. A corresponding value reduced to 20°C was 0.293; and for 25°C, .365. These values of K_r seem reasonable for what should be expected in such a stream, where the major sources of pollution are some distance above. These values were used to calculate what total organic loading must have entered the stream or been picked up from the bed, in addition to the recorded effluents, to cause the total pollution load observed at Little Falls:

The following are the results obtained for 1967, in lbs./day of ultimate BOD.

TABLE 1-1

UPPER PASSAIC RIVER ORGANIC LOADINGS

	Lbs. BOD Ult. Per Day
Organic loading removed from effluents . . .	101,400
Organic loading entering stream in recorded effluents . . .	27,500
Background organic loading*	22,960
Other man-made organic loading*	35,650
Total organic load entering the river system . . .	86,090

These results indicate that besides the organic load entering the stream from recorded effluents, there is a load over twice as great due to other, unrecorded sources.**

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*Division between these two categories is approximate.

**Although the percentage of unrecorded BOD is high, comparable percentages were found in the two other major basins which were examined, the Upper Raritan and the Millstone.

Based upon rough estimates, only about 38% of this unrecorded organic load can be accounted for as "background pollution," originating in natural soils and vegetation. The balance presumably reflects storm runoff from streets and business establishments, unauthorized and minor effluents, agricultural activities, etc.* The data in Table 1-1 indicate that mean removal of BOD in treatment plants during 1967 averaged 78.7%, which is higher than that of most rivers elsewhere.

Considering figures similar to those of Table 1-1 over an eleven-year period, 1957-1967, there is considerable variation in annual total loading, but only a moderate trend upward in total organic river loading. In general, BOD concentration in the river at Little Falls diminishes generally with discharge, as shown in Figure 1-4. The concentration of BOD for a given discharge was somewhat less in 1967 than in any of the preceding years, and in the summer of 1968 in the Pine Brook area relative concentrations were lower still. These results may have been affected by the fact that 1967 was a year of high flow, and that in May 1968 there occurred the greatest flood in 30 years.

For purposes of the aeration experiments, a system of intensive data collection was set up for the reach Pine Brook-Two Bridges for use during the summer and early fall of 1967 and 1968. These operations are described in the next chapter.

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*No inference is intended that this very large amount of man-made but unrecorded pollution represents laxity in enforcement of water quality controls, since this basin has been well monitored by efforts of the Passaic Valley Water Commission, as well as the state Public Health Department.

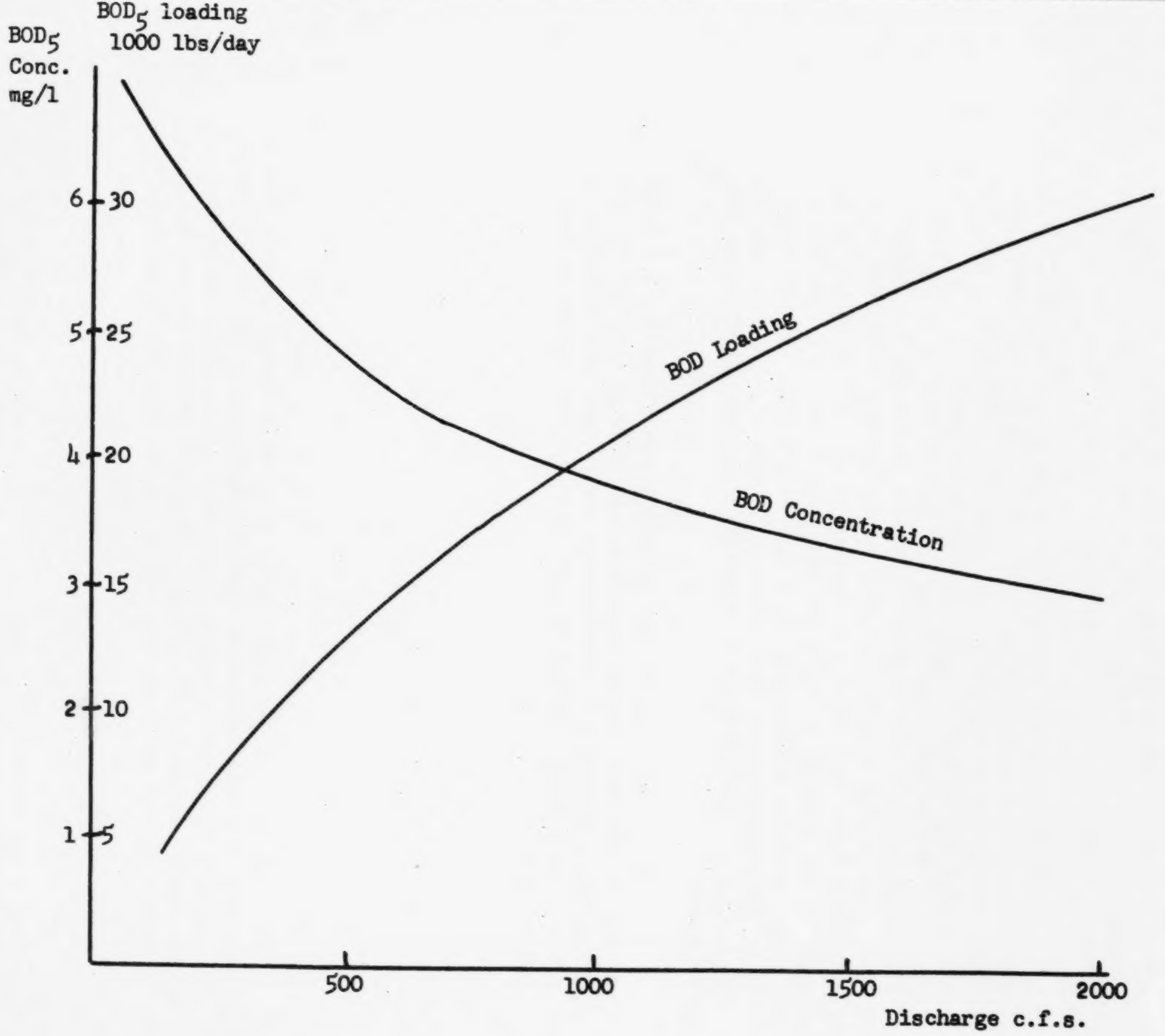


FIG. 1-4

VARIATION OF BOD WITH DISCHARGE,
LITTLE FALLS

APPENDIX I-1

HYDROLOGY AND HYDRAULIC GEOMETRY

Flow Characteristics

In Chapter I it is stated that the reach between Pine Brook and Two Bridges on the Upper Passaic River was chosen for performing the induced aeration operations. To design the aeration system by dynamic simulation, it is necessary to obtain flow frequency and hydraulic geometry data for this reach of the river.

Limited streamflow data at Pine Brook, which is near the aeration site, have been collected by the U. S. Geological Survey since 1963. In order to obtain a reliable flow duration curve for Pine Brook, the available short-term records needed to be adjusted to long-term records. The method used is the "index-station" method adopted by the U. S. Geological Survey as a standard procedure.(9) In this method a relation is established between the concurrent records of the short-term station and the "index" station, at which a long-term record is available. The flow-duration curve for the short-term station can then be derived from the flow-duration curve for the index station, assuming that the relationship holds for all frequencies.

There are two long-term stations which are suitable as good index stations. Chatham is about 14.6 miles upstream of Pine Brook and Little Falls, 15.2 miles downstream. Continuous discharge records at Little Falls are available since September 1897, and at Chatham since October 1937. The long-term flow duration curves for both stations have been prepared by the U. S. Geological Survey.(10)

Based on the published streamflow data by U. S. Geological Survey,(11) and also data collected during the present aeration study, correlation curves were constructed between discharges at Pine Brook and Chatham (Figure 1-6), and Pine Brook and Little Falls (unadjusted) (Figure 1-7). Flow-duration curves for Pine Brook were then derived from the long-term curves for these two index stations. The results (not shown graphically) show that there are no significant differences between the two derived frequency curves for low and medium flows. Only at high flows, the differences become apparent, as expected. Consequently the flow-duration curve derived from the longer record (1898-1960) at Little Falls were used, as shown in Figure 1-5.

The flow-duration curve mentioned above is on a year-round basis. It is clear that during summer months the flows are generally low, and the average distribution of flow to be expected during a long period does not apply to these months, during which the induced aeration is particularly necessary. Further discharge data for Little Falls from 1941 to 1967 were then obtained from the U. S. Geological Survey(11) and New Jersey Water Policy and Supply Division.(12) Flow duration curves for Little Falls for months from June through September were then constructed and it was found that frequencies for the four months were quite similar. The composite curve for Little Falls for June through September, and similar curve for Pine Brook are shown on Figure 1-5.

It is observed from these graphs that, at Little Falls, a discharge of approximately 670 cfs is on the average exceeded 50 percent of the time for the year round curve, but is exceeded only 12 percent of the time during summer months. Similarly for Pine Brook a discharge of 370 cfs at 50 percent exceedence level is only exceeded 12 percent of the time in the summer. At 50 percent exceedence level, the summer flow-duration curves indicate a discharge of about 120 cfs for Pine Brook and about 200 cfs for Little Falls.

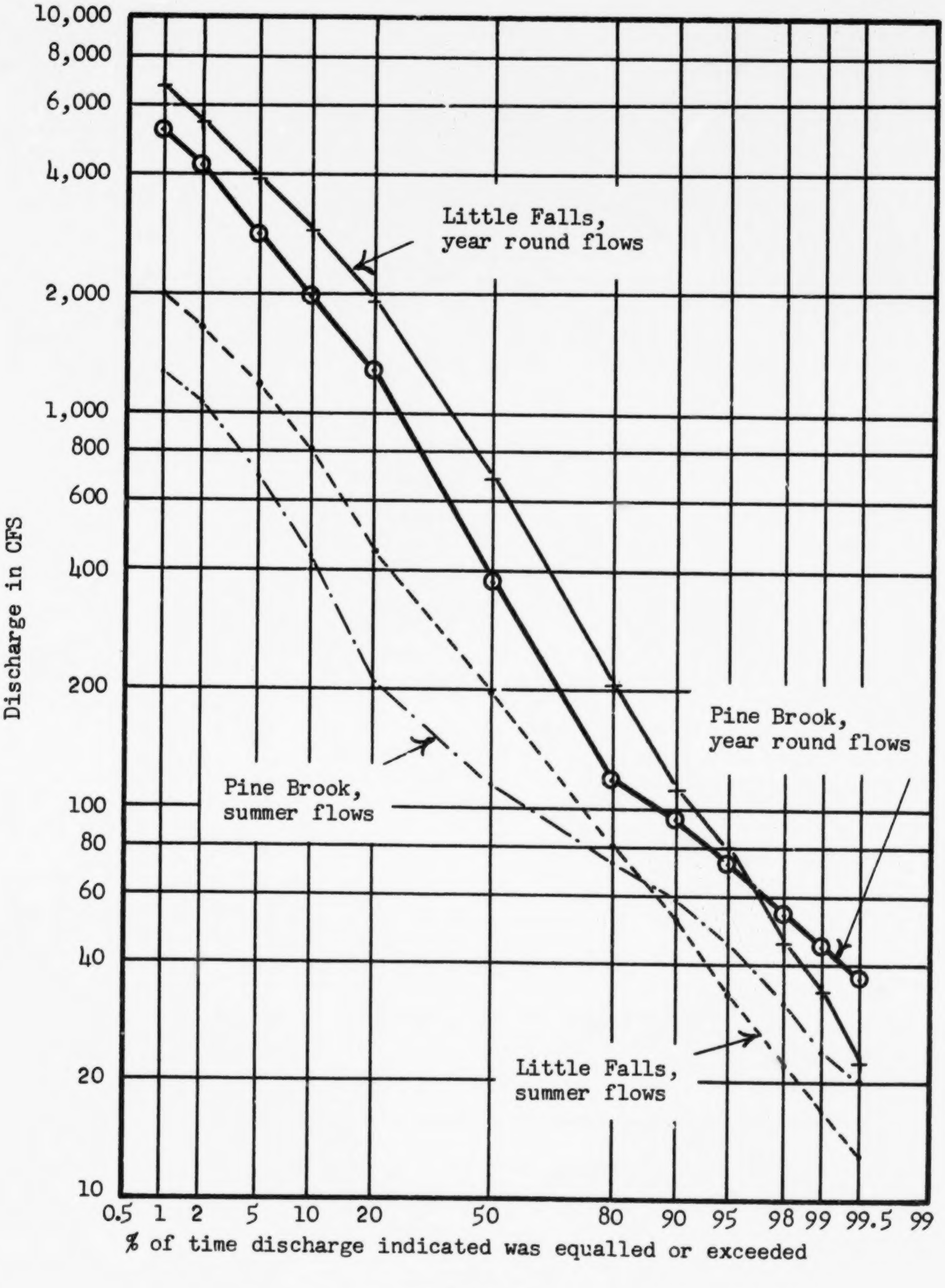
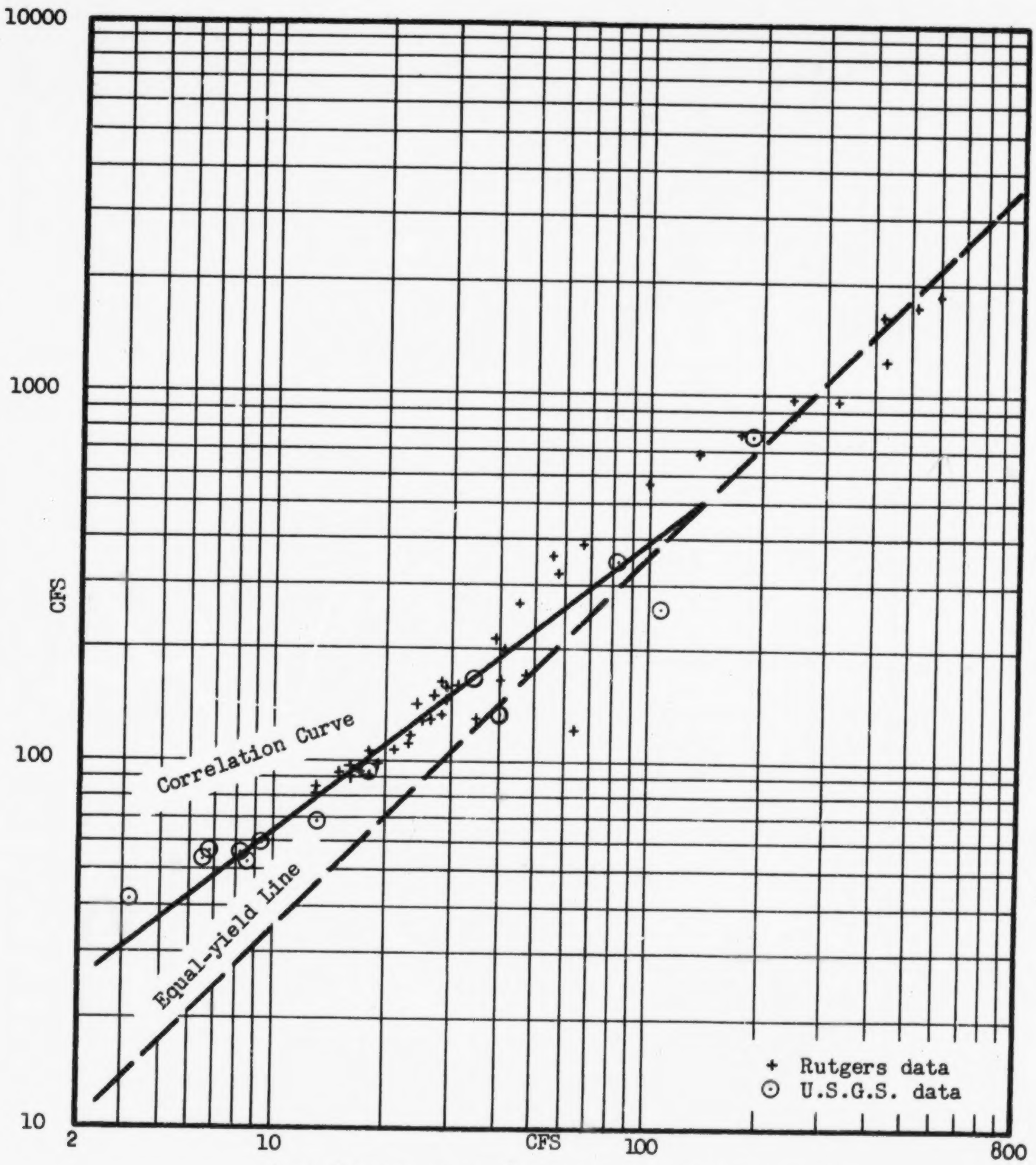


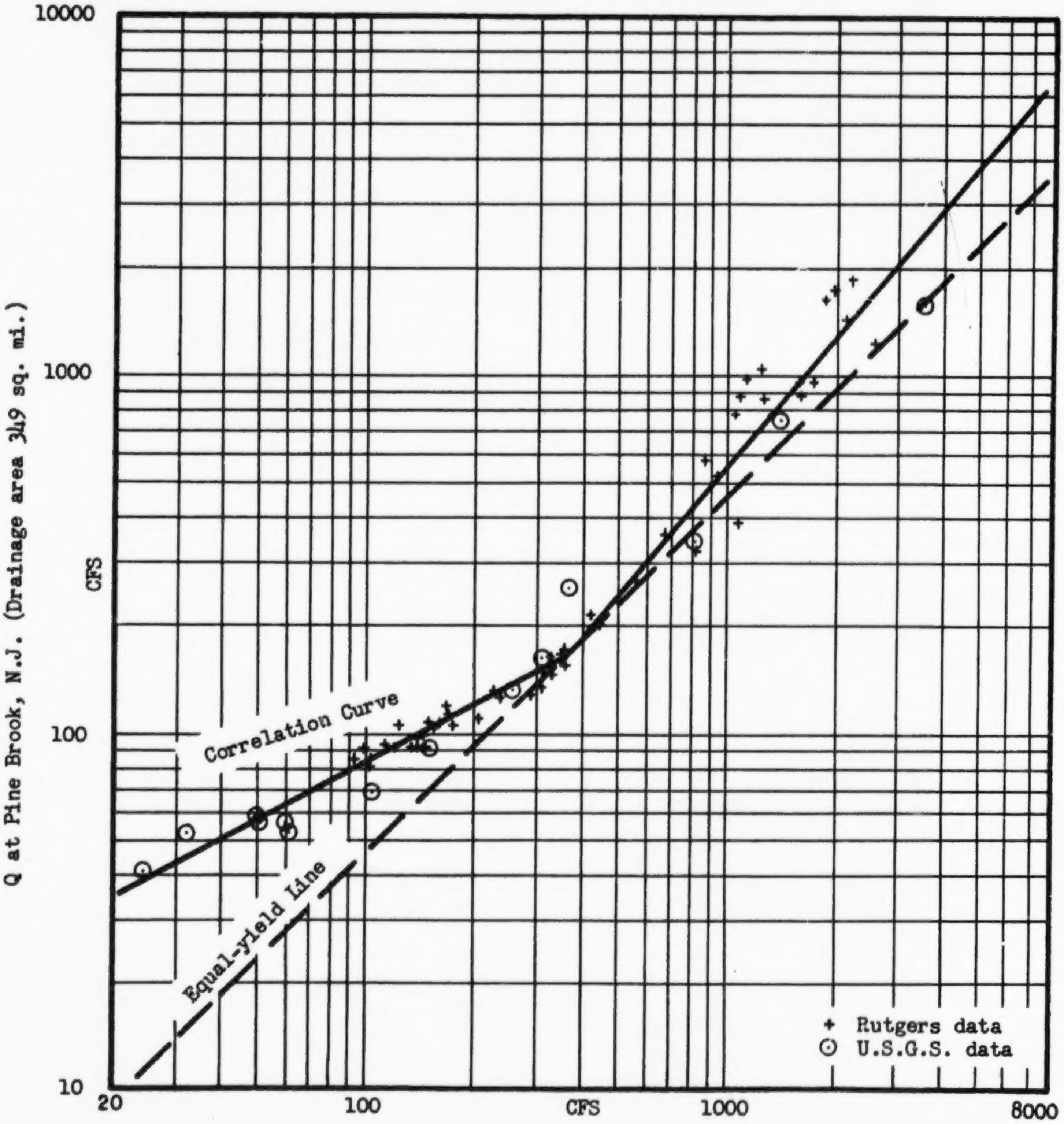
FIG. 1-5

FLOW DURATION CURVES, LITTLE FALLS
AND PINE BROOK ON PASSAIC RIVER, N.J.



CORRELATION CURVE BETWEEN THE DISCHARGE AT CHATHAM, N.J. AND PINE BROOK, N.J.

FIG. 1-6



CORRELATION CURVE BETWEEN THE DISCHARGE AT LITTLE FALLS, N.J.
AND PINE BROOK, N.J.

FIG. 1-7

Hydraulic Geometry

The hydraulic characteristics of the river reach within which the aeration operations were performed are of importance to the study of aerator efficiency and design. Therefore the relations among discharge, depth, and velocity at the aeration site were studied. Use was made of the data collected during the summer of 1968. The discharge data were obtained by using U.S.G.S. rating curve with observed gage readings. The water width and area were determined from the depth and depth-velocity traverse records. With these data the average velocity and the average depth for the cross section corresponding to a particular discharge could be determined.

It was proposed by Leopold and Maddock(13) that the relation of discharge to other hydraulic factors (e.g. depth and velocity) at a point on the stream could be described by the expressions -

$$H = a\ Q^{b} \qquad \text{.... Eq. 1-2}$$

$$V = c\ Q^{d} \qquad \text{.... Eq. 1-3}$$

where Q is discharge, H and V represent the average depth and average velocity, respectively. The letters a, b, c, and d are numerical constants to be determined empirically.

For the Passaic River data there was a total of 40 cases of discharge, depth, and velocity observations. They are grouped according to location as the following:

Location	No. of Cases
- 200'	3
- 100	8
- 73	4
- 69	6
0	3
+ 81	14
+ 100	2
Total	40

Because of reliability considerations, only three sets of these cases were studied, namely, at -100', at -73' and -69' (combined, since only 4' apart), and at +81', with a total of 32 cases. A linear regression analysis was performed, after using logarithmic transformation of the data, on Rutgers' IBM 7040 computer. Since in this case the simple correlation coefficient gives only the degree of association between log Q and log H or log V, rather than Q and H or V, the index of determination (ID) was used instead. The ID is defined as:(14)

$$ID = 1 - \frac{\sigma e^2}{\sigma d^2} \qquad \text{.... Eq. 1-4}$$

where σe^2 is the error variance, and σd^2 is the variance of the observed dependent variable. Thus the ID gives the percentage (or portion) of the variance in the dependent variable being accounted for by the regression model. Since actually values of Q, H or V, and H (estimate for H) or V were used, rather than logarithmics of them, to compute σe^2 and σd^2, the ID provides a better index of the goodness of fit of the regression model.

The computed values of a,b,c, and d also ID, for each of the three locations are given in Table 1-2.

TABLE 1-2

RESULTS OF REGRESSION ANALYSIS OF EQ. (1) d (2)

Location	Q and H	ID	Q and V	ID
-100'	$H = 0.390\ Q^{0.412}$	0.44	$V = 0.019\ Q^{0.626}$	0.49
-69'd -73'	$H = 0.408\ Q^{0.423}$	0.87	$V = 0.030\ Q^{0.520}$	0.90
+81'	$H = 0.697\ Q^{0.372}$	0.88	$V = 0.032\ Q^{0.468}$	0.89

It can be seen from Table I that, except for location -100' which had the fewest points, the equations seem to fit the data extremely well. It can also be seen that generally a and c increase, while b and d decrease in the downstream direction.

Equations (1-2) and (1-3) are commonly employed for the empirical determination of the coefficient of reaeration, k_2 of streams. Since it has been noted$^{(15)}$ that k_2 varies with discharge at any location, but also varies along the river reach. (k_2 decreases in the downstream direction at the 0.43 power of the discharge), it seems to be practical to derive an "average" relationship among discharge, depth, and velocity for a river reach in question. Consequently a weighted average (according to the number of observations) was arbitrarily used to compute an "average" relation among Q and H or V. The results are as follows:

$$H = 0.530\ Q^{0.398} \qquad \text{.... Eq. 1-5}$$

$$V = 0.028\ Q^{0.524} \qquad \text{.... Eq. 1-6}$$

These relationships are derived for flows measured on the portion of the river in the vicinity of the mechanical aeration.

However, the mean velocity of the river for the reach Pine Brook - Two Bridges is considerably less, as shown by dye tests conducted by the U.S.G.S.$^{(13)}$ These results show that the reach Pine Brook - Clinton (3.2 mi.) had a velocity of 0.34 mph on the same occasion that the Clinton-Two Bridges reach (8.6 mi.) had a velocity of 0.20 mph. For the combined Pine Brook - Two Bridges reach, data are available for two discharges. For the exceptionally low discharge of September 1964, the time of travel was 75.9 hours, at a discharge of 39.5 cfs at Pine Brook. For a more normal low discharge of 136 cfs on June 1964, the time of travel was 52.6 hours, or a mean velocity of 0.328 ft/sec.

During the areration tests three velocities near the Pine Brook site were observed at discharges of about 130 cfs, these are 0.65, 0.44, and 0.30 ft/sec, with a mean of 0.463 ft/sec. Apparently, therefore, the mean time of travel over the reach Pine Brook - Two Bridges is about 0.328/0.463 or 71% of that existing near the aeration site for normal summer flows.

From an examination of aerial photographs, the reach Pine Brook - Two Bridges has some variations in width, with the narrower portions near the central part and most of the wider portions near the bottom. However, the mean width of the upper and lower sub-reaches appears to be about the same. As an approximation, it might be assumed that the mean width remains unchanged, thus equations 1-5 and 1-6 may be revised to obtain mean conditions for the entire reach as follows:

$$H' = 0.746\ Q^{0.398} \qquad \text{.... Eq. 1-7}$$

$$V' = 0.020\ Q^{0.524} \qquad \text{.... Eq. 1-8}$$

in which

H' = mean depth in ft

Q = discharge in cfs

V = mean velocity in ft/sec

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Chapter II

FIELD OPERATIONS

Site and Equipment

Project operations were directed from the mechanical aerator site (#1) at the intersection of U. S. Highway 46 and the Passaic River, near Pine Brook, N. J. See Map, Figure 1-2. At this point the office/laboratory trailer was installed and a storage building was erected to house the electrical control center, the continuous sampling equipment, and boats and other equipment when not in use. It also served as a storage building for much of the heavy equipment during the winter of 1967-68; and in the summer of 1968, it contained the continuous monitoring panel, described later. The diffuser aerator site (#2) was located below Horseneck Bridge, about 2-1/2 miles downstream from site #1. (See Figure 1-3.) See Figures 2-1 through 2-4 for views of the aerators.

Prior to each summer of work, a contractor was chosen through competitive bidding to install the aerator or aerators, to remove them after the test period, and to move them into winter storage. The mechanical aerator installation required a 240 - volt, 3 - phase electrical service for the 75 HP motor, and 110 - volt single - phase service for lighting, refrigeration, instruments, and

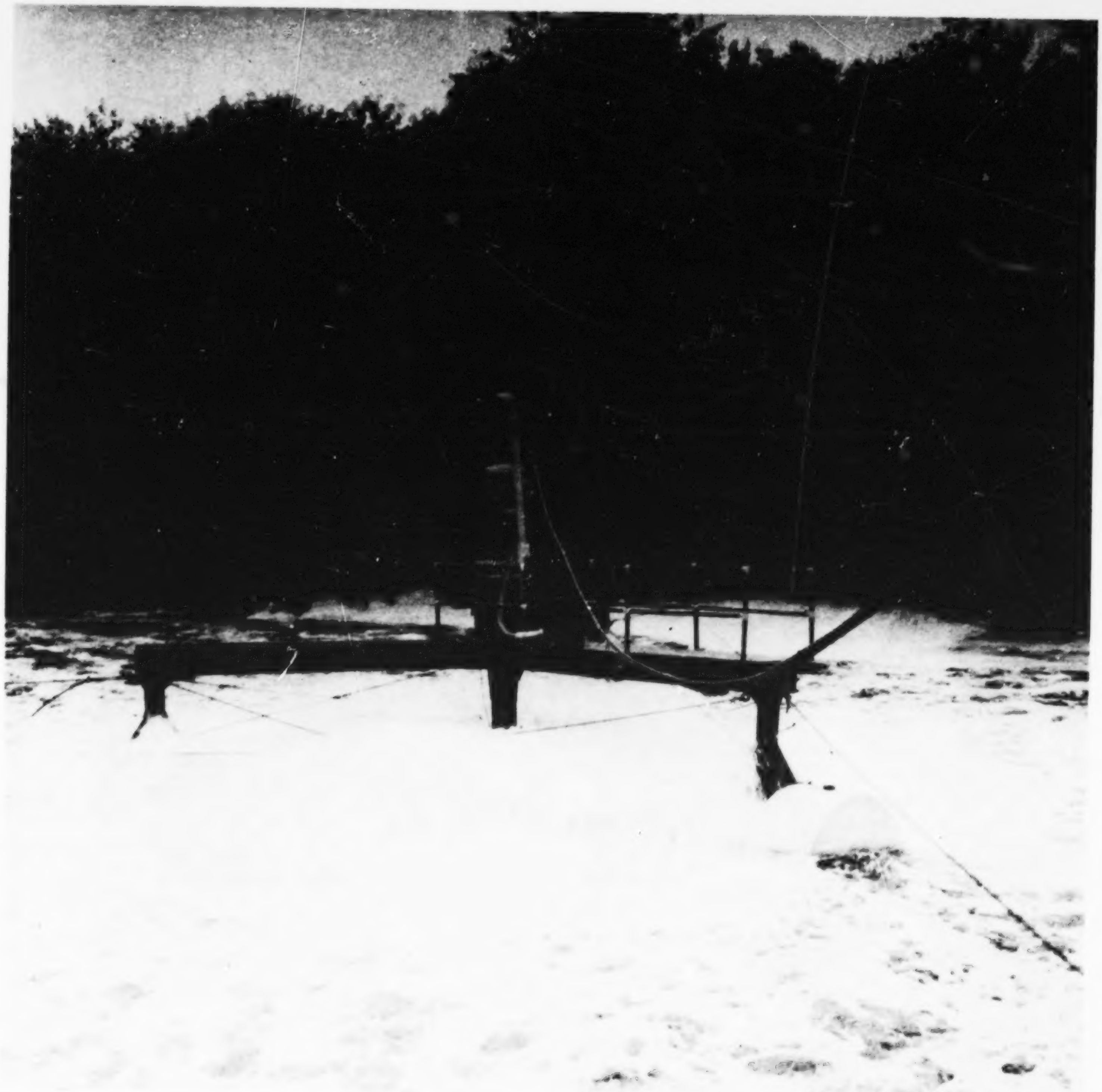

FIGURE 2-1

MECHANICAL AERATOR IN OPERATION

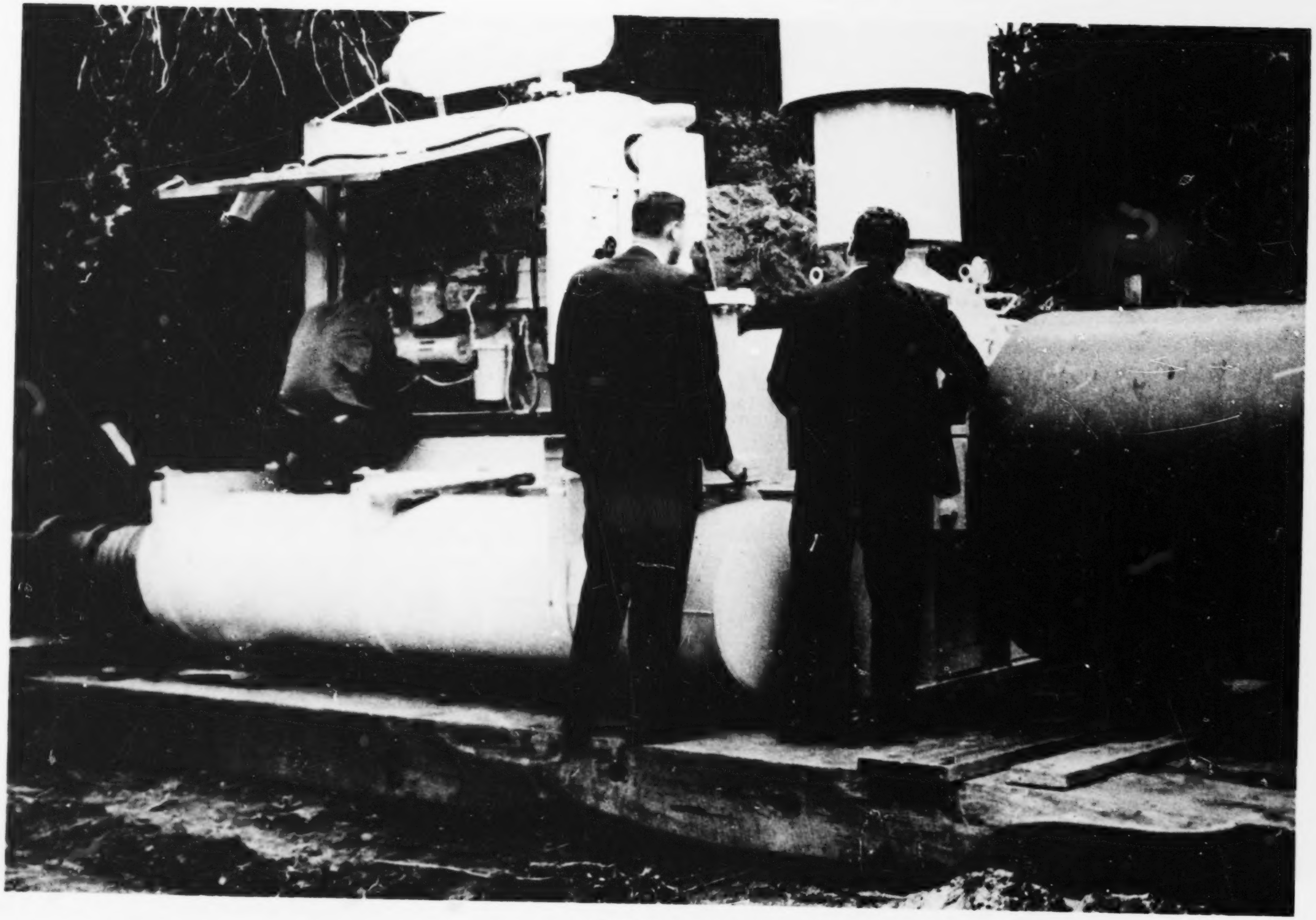

FIGURE 2-2

DIFFUSER AERATOR ENGINE, BLOWER, SILENCER AND FUEL TANK

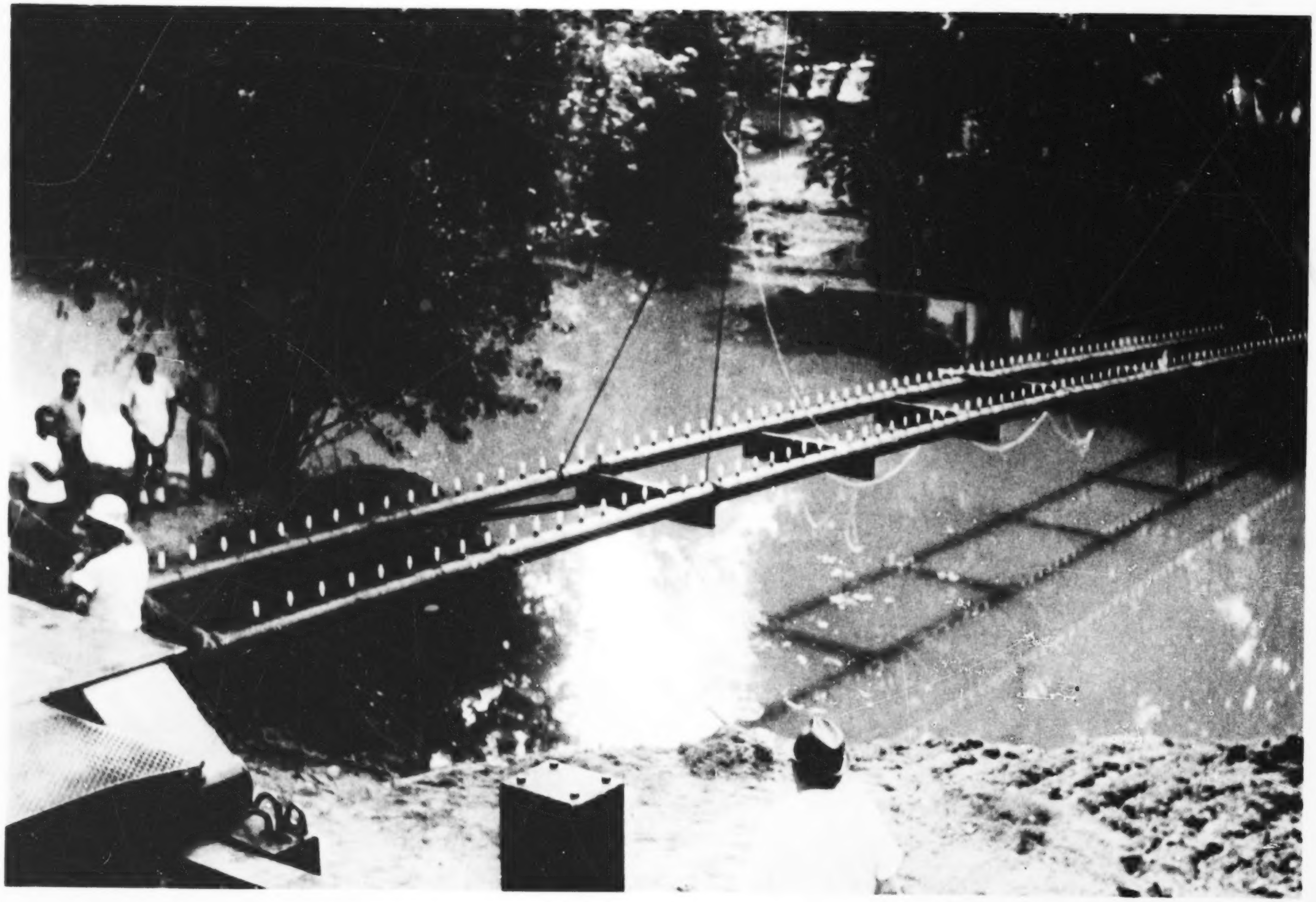

FIGURE 2-3

HEADERS WITH DIFFUSERS, READY FOR INSTALLATION

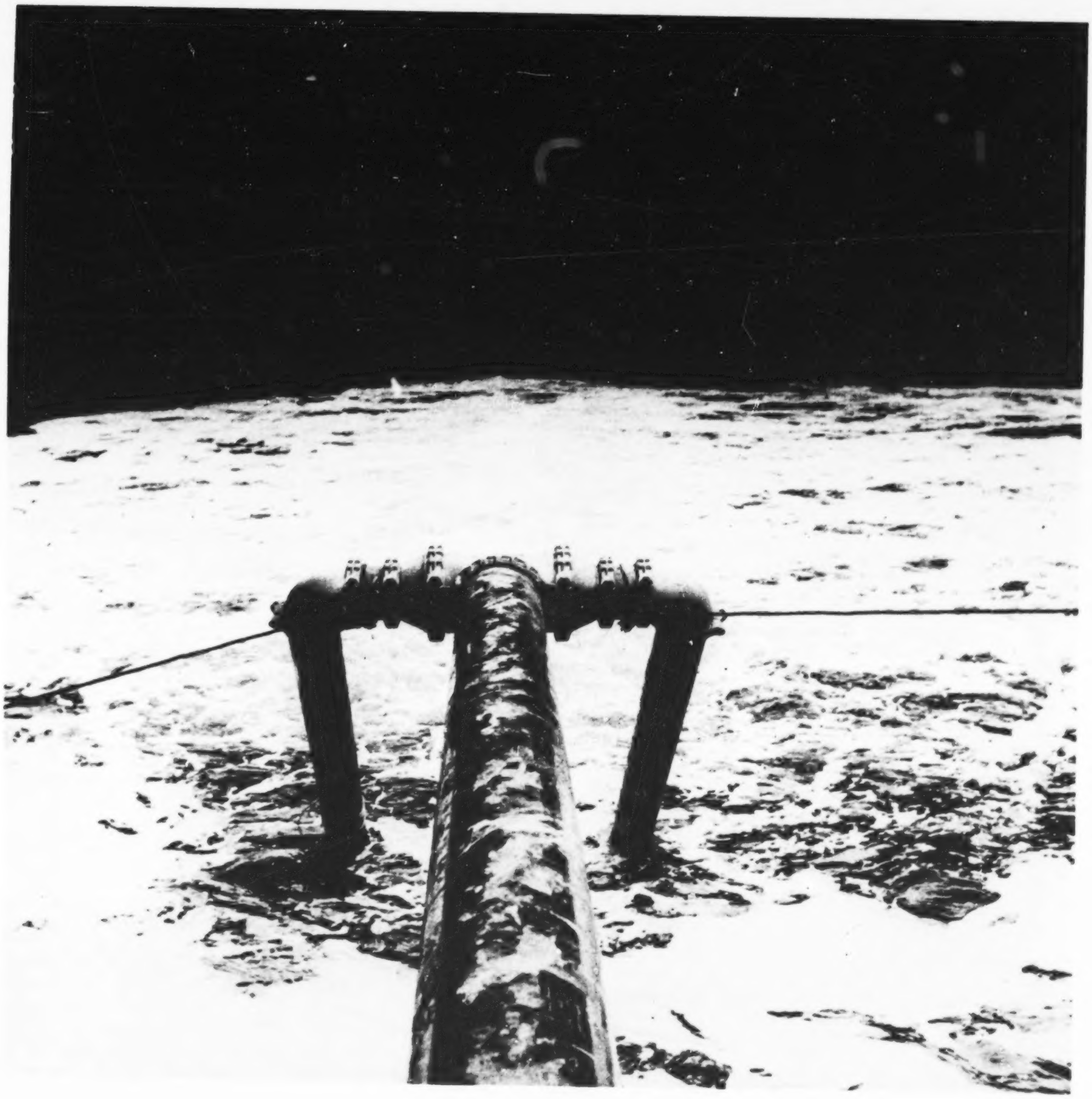

FIGURE 2-4

DIFFUSER AERATOR IN OPERATION

pumps. The diffuser aerator installation required clearing and leveling the site near Horseneck Bridge, placing the diesel compressor unit on a heavy timber foundation, placing two 8-inch diameter by 80 foot long air headers with diffuser nozzles on the bottom of the river, and connecting them to the compressor by means of a 12 inch air supply pipe. An orifice plate and manometers were also installed for measurement of the air flow rate.

The initial installation of the mechanical aerator was delayed by high water during June, 1967. Stream data were taken regularly throughout the summer, but the first data on the aerator were taken August 1.

Installation of both aerators was delayed by the occurrence of a major flood during May, 1968. The mechanical aerator become operational on June 28, and the diffuser aerator, the site for which was somewhat swampy, began operations on August 7.

Personnel and Operating Schedule

The general operations and testing procedures were carried out by students, supervised by Dr. Frank Dittman, the supervisor of field operations. Other tests of a special research nature were carried out by faculty and students engaged in that particular work. During most of each summer, operations continued 16 hours per day, 5 days per week. During the 1967 summer, there were normally two students engaged in making measurements and taking samples during the day shift and two during the evening, with the supervisor covering the day shift and part of the evening shift. The assistant supervisor and the fifth hourly worker maintained a flexible schedule, carrying on necessary work at the site or at the Rutgers laboratories, or in purchasing and altering equipment which they then delivered to the site. During parts of each summer, measurements and samples were taken 24 hours per day, by two workers per shift. During 1968 only, two additional students were hired, whose principal duties were to clean and maintain the continuous monitoring equipment described below.

Daily Procedure

During shifts when longitudinal traverses were scheduled, the two workers would check out their equipment, lock up and secure the building and trailer (if no one else was present), and then travel downstream in the boat either 3 miles or 12 miles, depending on the weather. They would then return upstream, taking oxygen meter readings, Winkler samples and BOD samples at various intervals, as recorded on the data sheets. Flow gauge readings were taken both before and after each traverse from the USGS flow gauge at Pine Brook.

After completion of the traverse, the BOD samples were stored in the refrigerator to await transportation to the Rutgers laboratories, since the field office was not equipped for BOD analysis. The Winkler samples had the fixing chemicals added in the boat, immediately after they were taken, to convert the oxygen content to an equivalent quantity of free iodine; our experience showed that this was necessary to obtain good results. After being brought back to the field office, they were titrated with sodium thiosulfate solution, and the resulting oxygen - content data were entered on the data sheets, for comparison with the readings of the portable oxygen meters.

On other days, as decided by the supervisor and/or the principal investigators, the workers made oxygen - velocity traverses across the stream, unsteady - state traverses, or circular traverses. These procedures are described in more detail below.

Miscellaneous Duties

In addition to the traverses and analytical work referred to above, other duties performed by the student workers during the summers included the following:

(1) Operation of aerator and collection of electrical data.

(2) Making depth traverses at various cross sections.

(3) Installing and maintaining distance markers near the aerators and 12 miles down the stream to Two Bridges, so that the location of each sample taken would be accurately known.

(4) Maintaining and operating the sample pumping system.

(5) Making sure that all data sheets were correctly made out in triplicate, and that required corrective calculations were made and entered.

(6) Making calculations and drawings on days when the weather did not permit outside work.

(7) Maintaining telephone communications with university offices, suppliers, and others.

(8) Keeping all laboratory equipment clean and in operating condition.

(9) Performing necessary maintenance work on other equipment as required.

(10) Maintaining cleanliness and security at the site at all times.

Longitudinal Coordinate System

Measurements were made along the banks of the river to establish stations for sampling and reference purposes every 1,000 feet from zero at site #1 down to station + 50,000 near Two Bridges (the junction with the Pompton River). The diffuser aerator was at Station + 14,200. Negative stations were upstream; the Pine Brook U.S.G.S. flow gage was at - 500.

Longitudinal D. O. Traverses

In order to determine the dynamic fluctuations of dissolved oxygen (d.o.) and biochemical oxygen demand (BOD), the above-mentioned longitudinal dissolved oxygen traverses were taken, usually starting at Two Bridges and moving up to station -500 in a period of about two hours. These traverses gave indications of the uptake of dissolved oxygen by the stream from the aerators. These data were valuable, but have the inherent defect that fluctuations are apt to be indicated by differences in conditions in the various water masses passed through. Intervals between samples were commonly either 1,000' or 3,000' (see Chapter IV).

Cross-Section D.O. - Velocity Tests

The most accurate measurements of dissolved oxygen in the stream were given by cross-traverses wherein meter readings of both d.o. and velocity were taken at 15 to 20 points at various positions and depths across the stream cross-section at one given longitudinal coordinate. The d.o. meters were calibrated by the Winkler method immediately before starting. The point velocities were measured by means of a Gurley current meter. Traverses of this type were usually taken 100 feet upstream and downstream of an aerator operating at steady state, as close together in time as possible. From graphical integration of these data, total oxygen input by the aerators could be determined. These data indicated that the one-foot depth centerline observations were quite closely characteristic of the cross-section (see Chapter IV).

Circular D.O. Traverses

On several occasions, circular traverses for d.o. measurement were taken around the mechanical aerator at various radii from 30' to 100', using a d.o. meter in the boat. The boat was maintained on station radially by means of a rope, and the angles were determined at 45° intervals by marks on the aerator. Due to the heavy turbulence, it was possible to take readings only at one foot depths, but nevertheless, interesting data were obtained. See Chapter IV for details.

Unsteady-State D.O. Tests

These tests were performed by placing two men with d.o. meters in boats anchored on the centerline at a known distance downstream from an aerator, 5 to 10 minutes before starting it up. Usually distances were +500 and +1,000 feet downstream. D.O. readings were taken at uniform time intervals of 1/2 to 1 minute, and continued while the aerator was started up and until it reached steady state in five to ten minutes, depending on the distance. The aerator was then stopped, and d.o. meter measurements were continued on the same time intervals until the effect of the aerator had completely disappeared. Meter readings were checked by frequent Winkler determinations, at the centerline, 1 foot deep. These data indicated closely the dynamic behavior of the stream-aerator system, and also gave accurate indications of the rate of steady-state oxygen input by the machines.

Flow Concentration Baffle Tests

In normal operation as shown by the circular d.o. traverses, the mechanical aerator draws in water from below and sends out highly oxygenated water in all directions from the center, including the upstream direction. As a result of its flow pattern, the aerator recycles and retreats much water, thus cutting down its efficiency.

To attempt to improve this situation, an inexpensive canvas baffle, 120 feet long by 10 feet wide, with a 10 foot by 13 foot hole in the center, was fabricated in 1968. By means of nylon ropes sewn into hems, the baffle was stretched across the stream and tied to trees. It was anchored to the bottom by 14 cinder blocks tied to grommets in the bottom hem.

When first installed, the baffle was only about 15 feet upstream from the aerator; also, the cloth tended to slide down the nylon ropes. The aerator threw its cascade of water entirely over the baffle, and there was danger of the canvas becoming entangled in the rotating impeller of the aerator.

Therefore, the baffle was removed, the ropes were anchored more firmly to the cloth and the baffle was reinstalled about 30 feet upstream of the aerator. It appeared that the concentration of flow through the baffle broke up the recirculation pattern to a considerable extent, and field personnel were of the opinion that conditions had been improved. However, only a few more observations were made before the canvas of the baffle began to disintegrate with exposure. When data were reduced to standard conditions, the results were inconclusive.

Move of Mechanical Aerator to Shallow Water

Because of the shallow water, both aerators were operated in basins excavated for the purpose in 1968. It was decided that it would be worthwhile to move the mechanical aerator from its basin, with an average depth of 7 feet at low water, to a new location where it could operate in natural depths of the stream, varying from 4 feet at the deepest point to zero at the banks. This was done late in the season. However, data obtained subsequently were few, and results were inconclusive.

24-Hour Floating Longitudinal Traverse

Most of the calculations relating to BOD-d.o. relationships on the Passaic River are made upon the assumption that the river was in a "steady state," with flow rate and basic pollution values constant. However, the continuous monitoring showed considerable short period variation in d.o., and some of the irregularities in BOD observed in longitudinal traverses were believed to result from differences in water masses travelled through during the traverse. Accordingly, a 24-hour test was organized in 1968, consisting of two sets of simultaneous observations, one taken at a fixed point above site No. 1 and the other from a boat floating down the river for 24 hours. The results conclusively demonstrated (1) the irregularity in BOD values passing a given point during 24 hours, (2) the much greater regularity of BOD samples taken from the boat floating down with the current, (3) variation of the floating sample BOD's with time, in accordance with expected relationships, and (4) photosynthetic variation of d.o. between day and night. For details, see Chapter VII.

Types of Data Obtained

During both summers on the Passaic River, the above-described activities resulted in the accumulation of voluminous files of data over the given 12-mile section of the river, under varying conditions of flow rate, weather, and sunlight, with aerators on and off. The types of data taken may be summarized as follows:

1. D.o. traverses, longitudinal
2. D.o. traverses, circular
3. D.o. and velocity traverses across the stream
4. BOD samples
5. Oxygen saturation determinations in river water
6. Aerator operating data
7. River depth profiles
8. D.o. observations, unsteady-state
9. Velocity traverses
10. Flow gauge readings
11. Instrument panel log sheets
12. Benthal oxygen demand data

River Conditions

The generally prevalent low flow rate of the Passaic River in summer, usually about 100 cfs at Pine Brook, had been one of the chief reasons the river was selected for the aeration study. The high discharge occurring during both summers interfered with the work, as mentioned above, but also resulted in some interesting data not obtainable under normal flow conditions. The mechanical aerator was operating during one period of very high flow rate of about 1,200 cfs. Such flows are very infrequent in the summer at this point (a discharge of 370 cfs being exceeded only 12% of the time in summer). At this high flow rate, the increase of d.o. concentration due to the aerators usually was less than 1 mg/l, but the input in pounds per hour was very high.

It was fortunate that these high flow rates occurred, since under this condition the transfer efficiency of the mechanical aerator was observed to be greater, and this helped to establish performance over a wide range of river conditions.

Monitoring Instrument Panel

In 1967, the only "recording instrument" used was a small sample pump for composite BOD samples, collected over a 24-hour period in a refrigerator. The river water was circulated continuously to the sample pump and back to the river by means of a hose, supplied by a small centrifugal pump on a raft.

In 1968 a much more ambitious instrument arrangement was decided upon, including the continuous indicating and recording of d.o., temperature and pH, and collection of a 24-hour composite sample at two different points, one upstream and one downstream of the mechanical aerator, at distances of 200 to 400 feet in each case. Each set of instruments was to be supplied with water from the indicated point by its own independent centrifugal pump installed on a raft located at the point.

Thus there were ten automatic instruments, plus pumps, hoses, and probe chambers to be kept in proper condition for 24-hour operation, if possible. Because of dirt and algae in the water, both of these recording systems quickly became fouled and required daily cleaning. The temperature and pH recorders and the samplers gave good results most of the time. The d.o. indicators and recorders, however, seldom gave reliable results, in spite of time-consuming efforts. The indicating and recording instruments on the same point frequently disagreed with each other and with the independent Winkler determinations. Therefore, the data obtained from continuous monitoring were not used in the quantitative analysis of aerator performance.

Mechanical Aerator Power Consumption

The electrically-driven mechanical aerator was provided with an instrument panel equipped with an AC voltmeter, ammmeter, and selector switches. The current in each of the three phases and all voltages between phases could be read over a short time interval, say 30 seconds. The shaft horsepower furnished by the motor* could be calculated from the equation

$$\text{Shaft H.P.} = \frac{\sqrt{3}\, E_1\, I_1 \,\text{Cos}\, \phi\, E_m}{746} \quad \text{.... Eq. 2-1}$$

where E_1 = arithmetic average of the 3 line voltages

I_1 = arithmetic average of the 3 line currents

cos ϕ = motor power factor**

E_m = motor efficiency**

During the summers of 1967 and 1968, there were 37 operating periods during which frequent records were kept. Shaft horsepower could therefore be calculated as a function of time, not continuously but at frequent intervals. The rated motor horsepower was 75; the highest calculated load was 105.0 and the lowest was 56.4. There were a very few occasions (perhaps three or four) when the motor stopped due to the opening of the overload breakers. The instruments indicated considerable variability in shaft horsepower with time during any given operating period. These variations occurred slowly, and of course were all limited to the above-mentioned range of 56.4 to 105.0 shaft horsepower. Figures 2-5 to 2-7, inclusive, show graphically some of the shaft horsepower variations with time for the 37 operating periods.

Power Variation with Time

In most cases the indicated shaft horsepower trend appears to be downward with time after starting--a trend

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*This is shaft horsepower applied to gear train, neglecting gear train losses.

**Furnished by manufacturer.

Curve	Date	River Flow Condition
a	7/26/68	Decreasing
b	7/25/68	Constant-maximum
c	7/22/68	Decreasing
d	7/23/68	Increasing

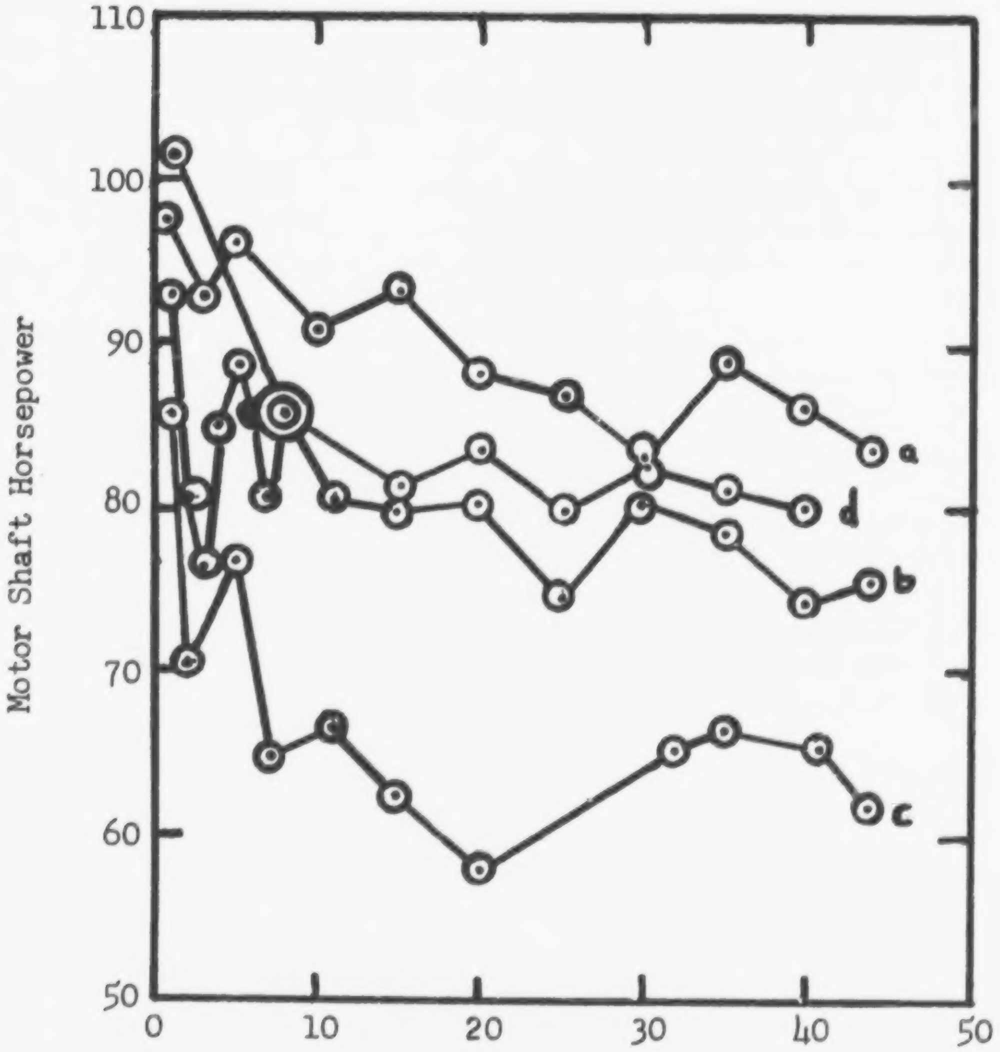

Figure 2-5

Mechanical Aerator

CURVES OF SHAFT HORSEPOWER VS TIME

0-50 MINUTE RUNS

Curve	Date	River Flow Condition
a	9/21/67	Constant
b	9/1/67	Decreasing rapidly
c	7/3/68	Decreasing

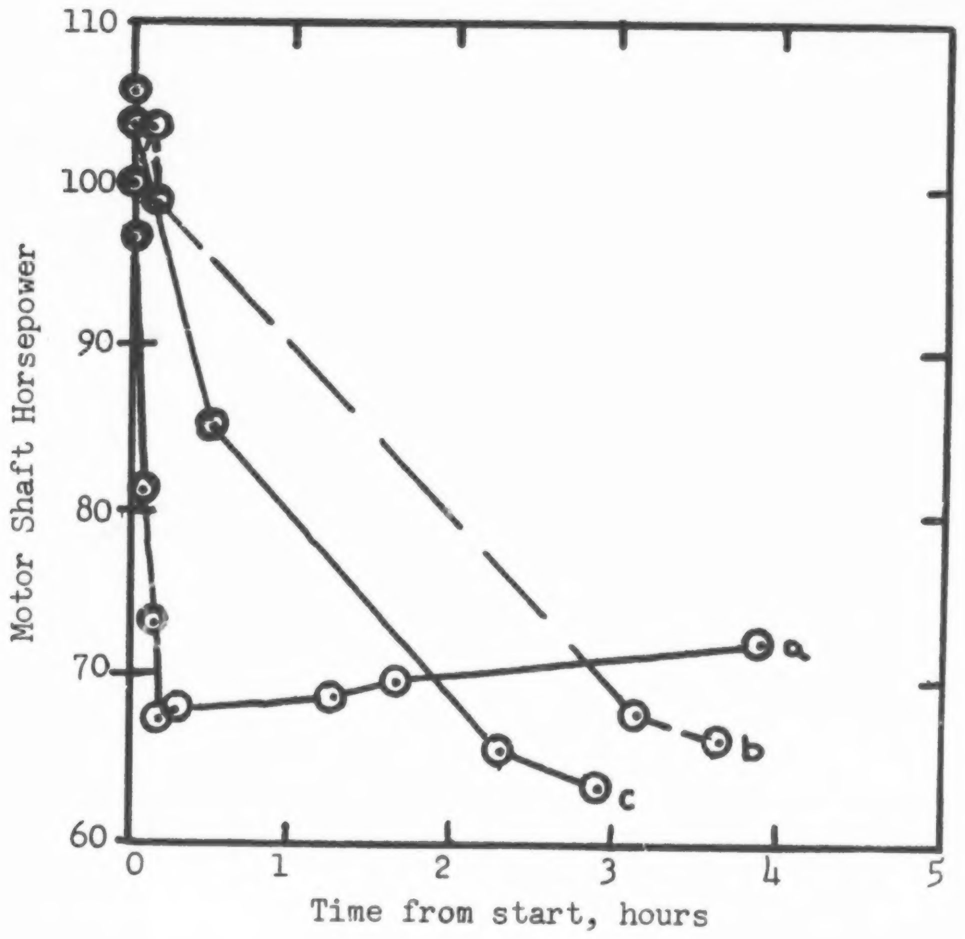

Figure 2-6

Mechanical Aerator

CURVES OF SHAFT HORSEPOWER VS TIME

0-5 HOUR RUNS

Curve	Date	River Flow Condition
a	8/1/68	Rapidly increasing
b	7/31/68	Increasing
c	8/10/68, 8/11/68	Slowly decreasing
d	8/23/68	Increasing

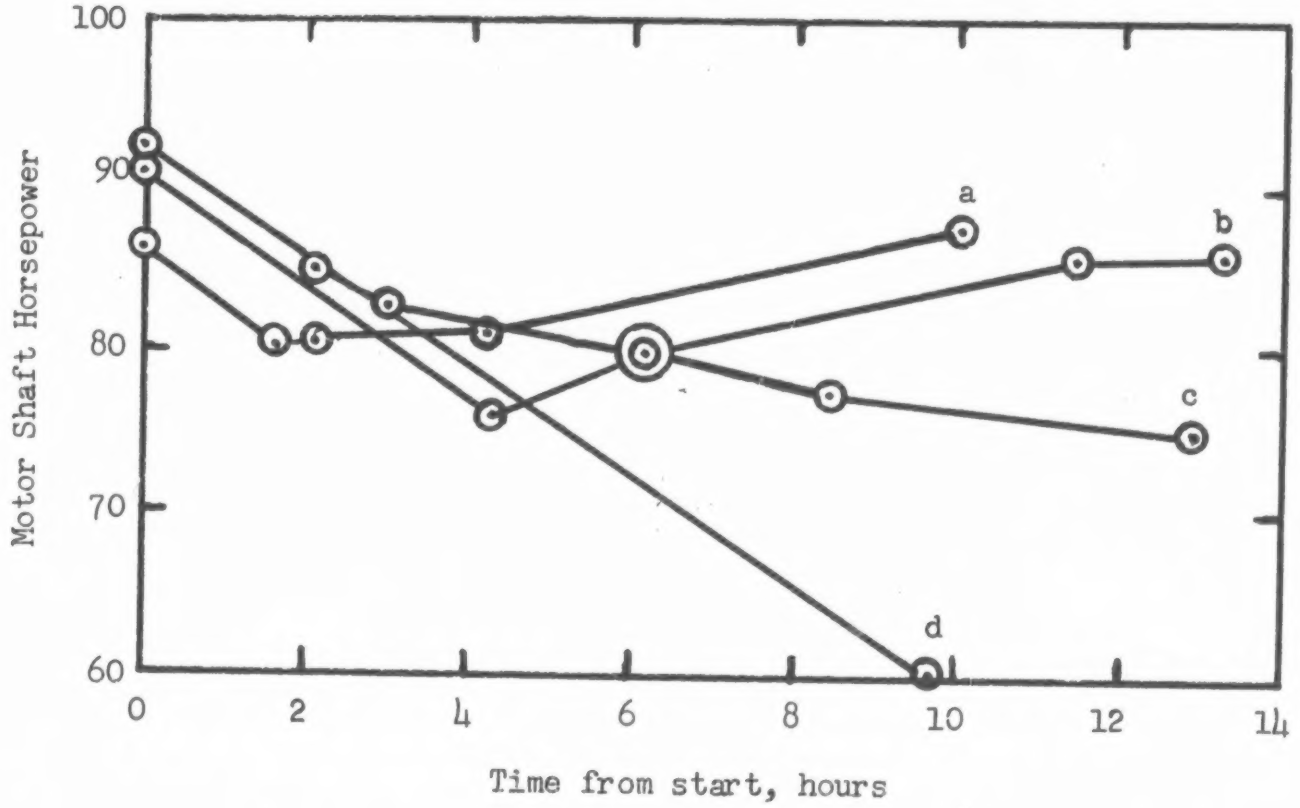

Figure 2-7

Mechanical Aerator

CURVES OF SHAFT HORSEPOWER VS TIME

0-14 HOUR RUNS

which continues for many hours, with occasional erratic increases. The cause is not definitely known, but a number of possibilities have been discussed:

(a) The motor and speed reducer, with their bearings and lubricants, might be slowly approaching thermal equilibrium with the surroundings. If this were so, the apparent decrease in aerator shaft horsepower would not be real; it would be due to mechanical inefficiences. This explanation does not seem very likely, because equipment of this type normally takes much less than one hour to reach thermal equilibrium, and the effect observed is too large.

(b) The water content of the entire excavated basin in which the aerator was located, about one hundred feet square, might gradually be put into slow rotation in the same direction as the impellor, thus decreasing slightly the force exerted by the blades in the water. This is oversimplified, but seems more plausible than (a) above.

(c) A complete analysis of how several principal variables change with time in the basin, such as bulk velocity at each point, vector directions of the bulk velocities, magnitude of turbulence at each point, and suspended bubble content of the water, might shed some light on this problem. However, most of the quantities required are difficult or impossible to measure with presently available equipment. Attempts made to measure the level of turbulence around the mechanical aerator were not productive because the intensity of the turbulence over-ranged the instruments and made impracticable the maintenance of the required firm, non-vibrating support for the turbulence probe.

(d) Visual observation of the aerator immediately after starting indicated a slight but definite downward movement due to the pumping action of the aerator, thus increasing the submergence, the power consumption, and the height of water thrown up in the cascade. However, it is not clear why this initial effect apparently reverses itself over a period of hours, decreasing the horsepower.

(e) In a few cases, the indicated power consumption of the mechanical aerator increased with time after the starting. These increases originally were thought to be associated with increasing stream flows, but careful analysis of the data did not support this explanation.

Operation of Diffuser Aerator

The source of air for the diffuser aerator was a Fuller Company Sutorbuilt rotary positive blower, #1216, series 3200, driven by a Cummins diesel engine through a Rockford clutch and reduction gear assembly, ratio 1.76 : 1.00.

The blower, gear assembly, and diesel engine were shop-assembled and tested for proper alignment and adjustment, and then delivered as a single unit. At the job site, the unit was supported on a heavy timber mat, resting on sandy soil. The location selected was about 200 feet back from the river and behind an earthen bank to minimize disturbance of the residents along the river due to noise. A 500-gallon diesel fuel tank was provided as a separate item, placed alongside the unit, and connected by means of a flexible tube arrangement. No fuel pump was required, since the fuel system of the Cummins engine provides enough vacuum to maintain flow after the system is filled.

After the engine had been put in operating condition by a qualified mechanic, the research workers were able to start the unit each day on battery power by pushing the start button.

The speed of the engine and blower could be varied over a considerable range by adjusting the throttle of the engine. Engine speed was indicated by a tachometer on the engine panel; the corresponding blower speed was equal to engine speed divided by 1.76. In making aeration runs, it was decided to use three specific engine speeds only, 1750, 1450, and 1150 rpm. The corresponding blower speeds were 994, 824, and 653 rpm.

Horsepower consumption of the blower was taken from the manufacturer's performance curve for the blower, which was prepared assuming air at an inlet condition of 70°F., 14.7 psia. Blower shaft horsepower was plotted vs. blower speed on four different performance curves, for discharge pressures of 6, 8, 10 and 12 psia.

Air from the blower was supplied to two 8 inch steel diffuser headers in the river by means of a 12 inch steel pipe. The 12 inch pipe divided into two 8 inch branches on the bank of the stream, above the water level.

In the 12 inch air supply line, about 70 feet from the blower outlet, an orifice meter was installed to check the performance of the blower.

The air-flow data from the orifice meter, when properly corrected for deviations of the air temperature and pressure from standard conditions, indicated that the blower followed the manufacturer's performance curve closely, with a correlation coefficient of 0.987.

Absorption Efficiency of Diffuser Aerator

During 1968, on three separate occasions, all required data were taken on the diffuser aerator to permit calculation of the transfer efficiency of the oxygen from the air bubbles to the water. Data needed were: corrected air flow rate, water flow rate, and increase of average oxygen concentration from upstream to downstream side of submerged aerator headers. It was assumed that dry air is 23.21% O_2 by weight. The d.o. observations were taken at stations 3,000 feet apart; and it is shown in Chapter IV (Figure 4-9) that actual transfer rates evidenced by readings close above and below the aerator average 20% greater than those taken at 3,000 foot intervals. The results below are adjusted for this relationship.

TABLE 2-1

Date (1968)	Air Flow Rate		Stream Flow Rate, CF/Second	O_2 Conc: In Water, ppm		O_2 In Air, lb/hr	O_2 Added To Stream, lb/hr		O_2 Absorbed by Stream, %
	SCF Sec	SCFM Nozzle		Upstream	Downstream		(obs.)	(Adj.)	
8-16	39.4	15.8	83.0	1.40	4.46	2515	57.0	68.6	2.71
8-19	33.8	12.6	106.0	0.25	1.75	2164	35.7	42.8	1.98
8-20	24.1	9.05	97.0	0.45	2.90	1538	53.2	63.8	4.15

The above results are quite irregular and much less than transfer rate specified by the manufacturer (6%), which, however, presumably referred to standard conditions.

APPENDIX II-1

ANALYSIS OF INSTRUMENT PANEL PERFORMANCE

As previously stated, the performance of the instrument panel used at Pine Brook during the summer of 1968 was disappointing and yielded little or no data of any value. The present section is written to present a quantitative discussion of the results and the problems encountered.

There were two identical sets of instruments on the panel; one set measured and recorded conditions in the water about 200 feet upstream from the mechanical aerator, and the other about 200 feet downststream. Two continuous streams of water, one from each point, were pumped to the instrument panel in the building by small centrifugal pumps on rafts. Plastic garden hoses were used as the conduits. The flow rate in each was about 10 gallons per minute.

Each set of instruments included the following:

One dissolved oxygen indicator

One dissolved oxygen recorder with strip chart and pen (using output signal from indicator)

One temperature recorder with electrically traced chart

One pH recorder with electrically traced chart

One automatic intermittent sampler, which accumulated a daily composite BOD sample in a refrigerated container

Each stream of water, upon reaching the instrument panel, passed through a plastic and metal "probe chamber" equipped with sensors or primary elements for sensing dissolved oxygen, temperature, and pH of the water. The probe chambers were designed to obtain water velocities of about two feet per second past the probes. Each stream then passed through its intermittent sampling tee and then through another hose back to the river.

Equation for Coefficients of Correlation

In comparing the panel instrument readings with independent check values obtained by other means, the results were evaluated by computing a correlation coefficient from the standard equation.(1)

$$r = \frac{\Sigma_i (x_i - \bar{x})(y_i - \bar{y})}{\sqrt{\Sigma_i (x_i - \bar{x})^2 (y_i - \bar{y})^2}} \quad \ldots\ \text{Eq. 2-1}$$

where

r = coefficient of correlation

x_i = ith panel instrument reading

y_i = ith corresponding independent reading

$\bar{x}$ = mean value of panel readings

$\bar{y}$ = mean value of independent readings

For a perfect correlation, $r = 1.00$; for no correlation, r = zero. A negative value of r indicates that the variable may be related reciprocally rather than directly as originally assumed.

Dissolved Oxygen Indicator Results Vs. Independent D.O. Measurements

Since oxygen traverses were being made daily in the river, using Winkler analysis and portable d.o. indicators (calibrated frequently), plenty of data were available for comparison. Thirty-five independent values were obtained and compared with what the upstream d.o. panel indicator had shown at nearly the same time and location; also, thirty-five data points were obtained and compared with the downstream panel d.o. indicator. The upstream panel differed from the independent check values by amounts ranging from +11.4 to -2.9 mg/l dissolved oxygen. Seventeen of the panel indicator values were high, and 18 were low. Four of the points were extremely far off, indicating some malfunction of the instrument. The coefficient of correlation was -0.161. Upon discarding four points which were extremely far off--far more than five times the standard deviation--a somewhat better picture emerged. Thirteen points in the panel instrument were high, eighteen were low and the deviations ranged from +0.8 to -2.9 ppm. The revised coefficient of correlation was 0.495.

Correlation of Data for all D.O. and Temperature Instruments

Proceeding in the same manner with upstream and downstream instruments for d.o. and temperature measurement, the results shown in Table 2-2 were obtained. With the exception described above, all of the correlation coefficients were positive, ranging from +0.495 to +0.878. The majority of dissolved oxygen values read on the panel were lower than the independent check values; the majority of the temperatures were slightly higher. Both of these factors could have been due to the pumping system. Algae, known to accumulate in the hoses, would use up oxygen in the absence of light. The environment outside the stream was normally warmer than the water, thus resulting in a slight heating of the water as it traveled through 200 feet of hose. Fluid friction also would contribute a small amount of heat.

Correlation of Data for pH Recorders

Data from the pH recorders could not be analyzed statistically, since few independent readings were taken. Those readings which were taken agreed with the panel instrument within 0.1 or 0.2 pH units. The workers who maintained the panel instruments calibrated the pH recorders daily with standard buffer solutions. There were no chemical effects in the pumping system to cause the pH

TABLE 2-2

SUMMARY OF RESULTS

INSTRUMENT PANEL - PASSAIC RIVER PROJECT - 1968

Name of Instrument	Source of data for Comparison	Total No. of Points	No. points (1) higher than Comparison data	No. points (2) lower than Comparison data	Maximum high Deviation	Maximum low Deviation	Coefficient of Correlation
D.O.Indicator upstream	Portable d.o. meter and Winkler d.o. determination	35	17	26	(11.4 before dropping) .8 after	-2.9	-0.161 (all data) +0.495 (dropping 4 bad points)
D.O.Indicator downstream	"	35	4	37	.3	-4.6	+0.692
D.O.Recorder upstream	D.O.Indicator upstream	30	15	9	.3	- .7	+0.864
D.O.Recorder downstream	D.O.Indicator downstream	23	10	13	2.7	- .8	+0.745
Temperature recorder, upstream	Temperature scale on portable D.O. meters	15	10	2	5.5	-1.1	+0.878
Temperature recorder, downstream	"	30	27	3	6.0	-3.0	+0.721

Notes: (1), (2): The sum of these may not equal the total number of points. Some of the points were in agreement.

readings at the panel to differ appreciably from their values in the stream. The pH records are undoubtedly the most accurate records obtained from the instrument panel. The Passaic River at Pine Brook, during the period of this recorder study in 1968 remained at all times at a pH within a few tenths of 7.0, the neutral point.

Discussion

The coefficients of correlation obtained by the comparison of d.o. and temperature records with independent check values are not valueless, from the statistical point of view. At the same time, it must be recognized that statistically significant data are not necessarily of scientific value. Quantitive d.o. results, for example, if used in computation of aerator performance, should be reliable within ± 0.1 mg/l; if used to evaluate stream analog simulation results, they should be good within ± 0.2 mg/l. Temperature data with ± 0.5°C ought to be obtainable regularly, partly because the saturation value of d.o. varies quite rapidly with temperature, but also because biological processes are generally temperature sensitive. It was fortunate that the direct instream measurements made from the boats yielded data of the required quality, since the panel instruments did not.

Comparison of Portable Meter Readings With Winkler Analyses

The portable meters were calibrated by means of the Winkler method at the beginning of each working day, and sometimes more than once a day. The number of pages of instream d.o. determinations was too great to permit calculation of a coefficient of correlation for every page. It was obvious by inspection, however, that the Winkler d.o. determination agreed closely with the Delta portable d.o. meters.

Selecting three pages of data at random, and using the Tippett equation referred to above, coefficients of correlation of 0.995, 0.972, and 0.985 were obtained.

Conclusions

For future recording instruments installations, the following suggestions are made:

a. Place instrument sensing devices directly in the stream; eliminate pumping systems.

b. Have instrument panel in sheltered location, with sensor cables long enough to reach from the panel to the desired sampling station.

c. Instrument sensor cables must have high-quality waterproof insulation to avoid degradation or loss of the small signals being transmitted. High-quality amplifying transmitters can be obtained, although they are rather expensive.

d. Several spare sensors of each type should be available, so that cleaning, repairing, and recalibration of sensors may proceed while continuous records are being obtained.

e. Water-proof plug-in jacks at the point of measurement should permit quick and reliable switching or substitution of sensors.

f. Electrically-traced charts seem to operate more reliably unattended than pen-and-ink arrangements.

Adoption of these recommendations should provide a substantial improvement. The potential advantage of continuous recording instruments is very great, if they can be made to work accurately without excessive time and cost for maintenance. The experience in this project was not sufficient to evaluate whether this is possible with presently available instruments.

REFERENCE

(1) Tippett, L. H. C., "Technological Applications of Statistics." (Wiley, New York, 1960), p. 151.

Chapter III

DISSOLVED OXYGEN DYNAMICS AND ANALYTICAL PROCEDURES

Introduction

The dissolved oxygen level of a stream is determined by the opposition of processes that tend to add or subtract oxygen. These forces produce what is usually described as an "oxygen sag curve," which is illustrated in Figure 3-1. This curve describes the depression of the dissolved oxygen level due to the initial dominance of the processes removing oxygen from the stream and a subsequent rise of the dissolved oxygen level, assuming that additive processes become dominant.

The sag curve, or rather its reciprocal, was initially described mathematically by Streeter and Phelps.(1) Their equation was:

$$D_t = \frac{Kd\ Lo}{Ka-Kd} (e^{-Kdt} - e^{-Kat}) + D_o e^{-Kat} \qquad \text{.... Eq. 3-1}$$

where

D_t = the oxygen deficit (C_s-C_t) in mg/l at time t

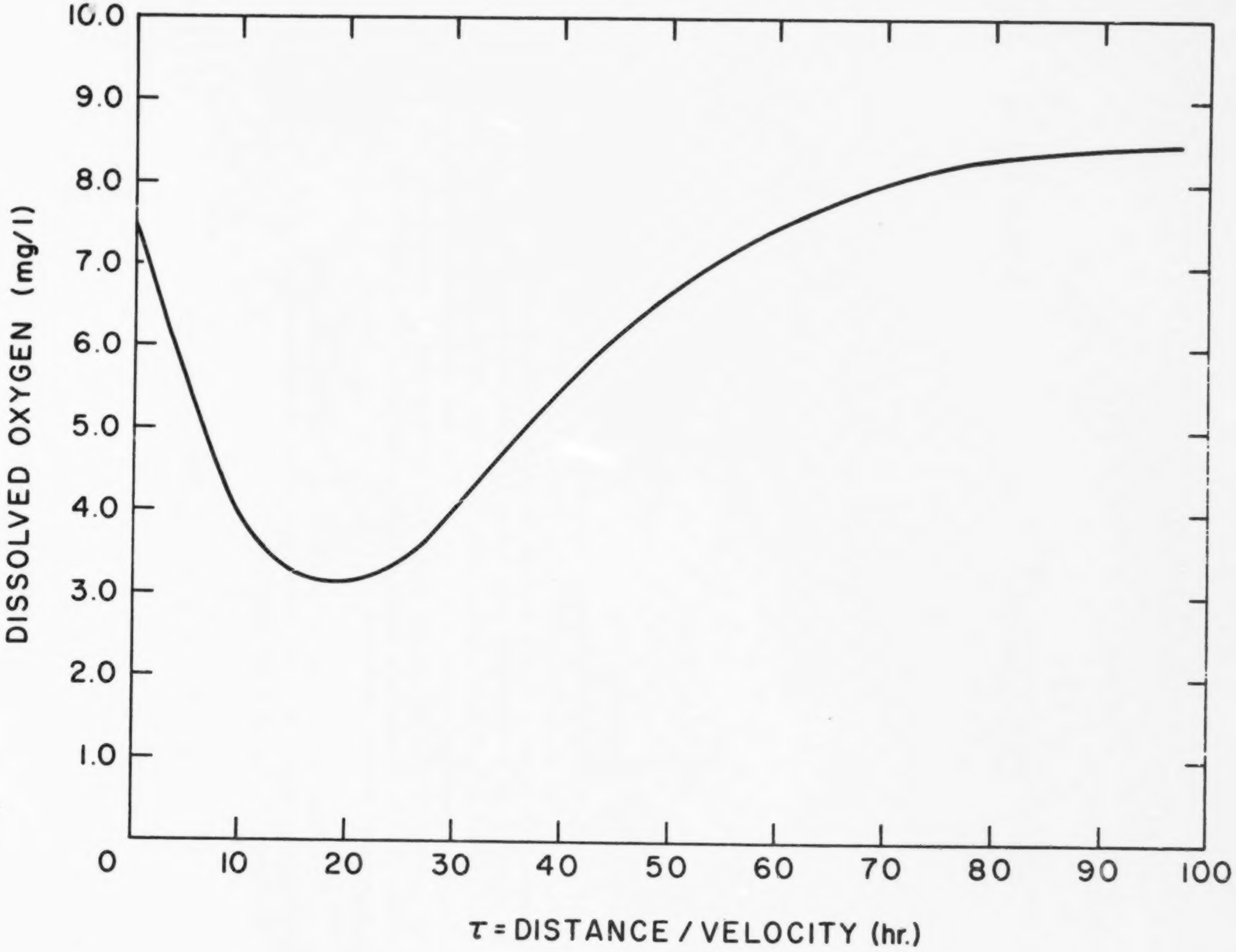

FIGURE 3-1. Generalized oxygen sag curve.

C_s = the saturation value of atmospheric oxygen in stream water in mg/l

C = the dissolved oxygen concentration in mg/l at time t

D_o = the initial oxygen deficit (C_s-C_o) in mg/l

C_o = the initial dissolved oxygen concentration in mg/l

L_o = the ultimate BOD in mg/l at initial time (t=0)

Kd = the deoxygenation constant due to oxidation of organic matter in the stream in reciprocal time units. It is equal to the fraction of the BOD remaining that is exerted per time unit (usually one day).

Ka = the atmospheric reaeration constant in reciprocal time units. It is equal to the fraction of the deficit remaining that is satisfied per time unit.

This equation describes the oxygen deficit for steady state conditions as being affected by only two processes. The deoxygenation is assumed due only to the oxidation of organic matter in the stream (BOD), and the reoxygenation is assumed due only to atmospheric reaeration.

Since the original formulation, there have been many modifications of this equation developed to include other processes that affect the dissolved oxygen level. Such processes include the dissolved oxygen contribution from photosynthesis, and dissolved oxygen sinks like benthal deposits and nitrification. An example of one of these equations, adapted after O'Connor,(2) is as follows:

$$D_\tau = \frac{Kd\ Lo}{Ka-Kr}\,(e^{-Kr\tau} - e^{-Ka\tau})$$

$$+ \frac{KnNo}{Ka-Kn}\,(e^{-Kn\tau} - e^{-Ka\tau})$$

$$+ D_o (e^{-Ka\tau})$$

$$+ \frac{S}{Ka} (1-e^{-Ka\tau})$$

$$- \frac{P-R}{Ka} (1-e^{-Ka\tau}) \quad \text{.... Eq. 3-2}$$

in which the new terms are:

K_r = the BOD removal constant for the stream, in which K_r = Kd + Kc, where Kc is the removal constant due to adsorption, sedimentation, etc.

N_o = the ultimate oxygen demand due to the oxidation of ammonia and nitrites in the stream (nitrification) in mg/l.

Kn = the deoxygenation constant due to the oxidation of ammonia in the stream in reciprocal time units. It is equal to the fraction of the nitrification remaining that is exerted per time period (usually one day).

τ = the distance downstream the deficit is measured (x) divided by the stream velocity (v) or x/v. This represents the time required to travel the distance x, and so is equivalent to t in the original equation.

$D\tau$ = the oxygen deficit at "time" τ

S = the benthal oxygen demand in terms of mg/l of oxygen consumed per time unit (day).

(P-R) = the net production of photosynthetic oxygen in terms of mg/l per time unit (day)

Artificial reoxygenation by mechanical or diffused air processes increases the dissolved oxygen level at or immediately below the areator. Once this effect has been produced, the natural stream processes determine the fate of this added oxygen. The importance of this or similar equations in the Passaic River artificial reoxygenation project lies in their ability to predict the new deficit or sag curve from the point of reoxygenation. This should allow the prediction of the persistence of the effect and the number and spacing of the aerators required to achieve some specified effect.

To use this latter equation, it is first necessary to determine the values of the equation parameters. These can be evaluated from field and laboratory studies, but due to both practical and theoretical considerations, all are not of equal reliability, which will be evident from the following sections describing the methods and results of the parameter evaluations. The parameters so evaluated were C_s, L_o, Kd, K_n, N_o, Ka, S, and P-R. The value of D_o is determined from the dissolved oxygen concentration at T_o and a knowledge of C_s. The predicted $D\tau$ value can also be converted to dissolved oxygen concentration through a knowledge of C_s.

The following sections indicate the studies made to evaluate the equation parameters, as closely as practicable, as a guide for subsequent sections. Particular attention was paid to the field estimations of benthal oxygen demand and photosynthetic reoxygenation, as it was suspected that these factors could play an important role in the oxygen balance of the Passaic River.

The Saturation Value C_s

The solubility of oxygen in water in equilibrium with the atmosphere is influenced mainly by temperature and salinity. Values for the saturation concentration C_s in units of mg/l are found in many texts such as "Standard Methods,"(3) usually is as function of salinity and temperature. Although these values, originally obtained around the turn of the century, are still widely accepted, there have been more recent studies by Truesdale et al,(4) Morris,(5) and the SED-ASCE.(6) All of these are different from the "Standard Methods" values.

At present, the values most used besides the "Standard Methods" values are those of the SED-ASCE. A comparison of these values are as follows for zero salinity.

Source	Temperature, °C			
	0	10	20	30
S.M., 10th ed.	14.62	11.33	9.17	7.63
SED-ASCE	16.65	11.27	9.02	7.44

The water of the Passaic River in the vicinity of the aerator contained organic pollution, and there is always a question as the effect of this on both the rate of reaeration and on the saturation value C_s. Theoretical considerations support the possibility of the former but not the latter. As it was uncertain which of the "standard" values to use even if no effect on C_s was assumed, a study on the Passaic River water was made to determine C_s.

A sample of river water was taken to the laboratory and placed in six, 4-1/2 foot by 1/2 foot cylinders. The water in the cylinders was deaerated, and the oxygen level measured as a function of time at a mixing rate of 0.36 cycles/minute. Biological activity was inhibited during this period as described by Rand.(7) The results obtained are found in Table 3-1. The saturation value averaged 9.25 for the six cylinders after equilibrium was achieved. This was at 19°C, which puts this value almost exactly between the "standard methods" and the SED-ASCE values for zero salinity. It can be concluded, therefore, that the organic pollution did not affect C_s, and that either of the two sets of values could be used, as they are similar over the temperature ranges of study. As the SED-ASCE values represent the most extensive recent work, it was decided that these should be used.

The saturation values are very temperature dependent. If the SED-ASCE values are employed, the temperature relationship is:

$$C_s = 14.652 - 0.41022T + 0.007991T^2 - 0.000077774T^3 \quad \text{.... Eq. 3-3}$$

where t = temperature in °C. As the dissolved solids concentration of the Passaic should not vary enough to change C_s, the salinity effect can be ignored.

Table 3-1

Oxygen Uptake Data for Passaic River Water

For a Water Quality of

BOD = 9.1 mg/l
TOC = 10.5 mg/l
COD = 16.8 mg/l

Time (hrs)	Dissolved Oxygen Concentration, mg/l					
	Cyl.1	Cyl.2	Cyl.3	Cyl.4	Cyl.5	Cyl.6
0	0.45	0.85	0.72	1.00	1.13	0.65
21	3.69	4.01	4.02	4.18	4.05	3.82
28	4.34	4.85	4.90	4.95	4.92	4.52
41	5.22	5.70	5.69	5.71	5.75	5.30
51	5.77	6.28	6.30	6.41	6.50	5.98
65	6.51	7.00	6.90	7.00	7.17	6.55
76	7.10	7.50	7.45	7.50	7.59	7.15
89	7.50	8.01	7.96	8.00	8.10	7.78
140	8.17	8.50	8.50	8.52	8.60	8.37
163	8.50	8.70	8.70	8.68	8.80	8.55
196	8.78	8.88	8.90	8.85	8.90	8.80
235	8.97	9.01	9.07	9.05	9.17	9.00
284	9.10	9.13	9.18	9.17	9.25	9.15
335	9.16	9.17	9.19	9.17	9.27	9.20
359	9.23	9.26	9.23	9.25	9.27	9.24
382	9.25	9.25	9.26	9.24	9.25	9.25
401	9.25	9.23	9.26	9.25	9.25	9.23

Atmospheric Reaeration and Ka

When a stream is deoxygenated either artificially or through the various biological oxygen consuming processes occurring within it, the oxygen so removed is replaced either by diffusion of oxygen from the atmosphere into the stream, or through the production of photosynthetic oxygen in the stream itself. Of the two sources of oxygen replenishment, atmospheric reaeration has historically been ascribed as being the most significant, and has been studied quite intensively.

The rate of atmospheric reaeration has been demonstrated to be proportional to the oxygen deficit "D," sometimes referred to as "driving force." This deficit is equal to the saturation minus the actual dissolved oxygen concentration, or "C_s-C." Mathematically, this can be expressed

$$\frac{dC}{dt} = K_a \ (C_s - C) \qquad \text{.... Eq. 3-4}$$

where

t = time

K_a = proportionality constant, or reaeration coefficient in reciprocal time units.

This equation may also be presented in terms of the deficit "D," or

$$\frac{-dD}{dt} = K_a D \qquad \text{.... Eq. 3-5}$$

To evaluate atmospheric reaeration in a stream, it is thus necessary to determine K_a, which is the fraction of the deficit satisfied per unit time.

The constant K_a can be determined in a stream using the integral form of equation 3-5. This is

$$K_a = \frac{\ln D_1 - \ln D_2}{\tau} \qquad \text{.... Eq. 3-6}$$

where

D_1 = the upstream deficit (station 1)

D_2 = the downstream deficit (station 2)

τ = the time between stations

Sometimes the $\log_{10}$ form of the equation is used, which is

$$k_2 = \frac{K_a}{2.3} = \frac{\log D_1 - \log D_2}{\tau} \qquad \text{.... Eq. 3-7}$$

To use this equation, all that is required is the knowledge of the temperature (to obtain C_s) and dissolved oxygen at two locations on the river. If there is steady state conditions, or if the same segment of river water is studied, the average K_a between the two locations can be calculated.

The upstream deficit is usually produced chemically by the addition of sodium sulfite and cobalt catalyst to the stream. After all sulfite has reacted

$$2\ SO_3^= + O_2 = 2\ SO_4^=$$

the dissolved oxygen concentration and temperature at the two locations can be measured. This method is best applied to waters in which there is little biological activity taking place. If the stream is biologically active, and the dissolved oxygen concentration is being influenced by the biological and benthal oxygen demands, nitrification, and photosynthesis, then its application is questionable. When used under such circumstances, the time between the two stream stations is reduced to minimize these other effects. It is doubtful, however, if their effect can be completely eliminated.

Another approach is to calculate K_a from the oxygen sag equation itself, or a modification applicable to the stream under study. The upstream and downstream deficits are easily determined for such use, but the K_a value so obtained would be subject to the errors and bias inherent in the determination of the other equation parameters. In a stream in which the biological processes noted previously are occurring, this can be a laborious and inaccurate process. This also puts the investigator in the position of using the equation to determine one of its parameters so that the equation can be used. A K_a value without such bias would be more appropriate.

The atmospheric reaeration constant K_a must be a factor of geometric as well as other considerations. The initial approach to evaluating K_a from stream geometrical, etc. considerations has been presented by Black and Phelps,(8) in which they stated

$$K_a = 2.3\ c\ \frac{V^n}{H^2} \qquad \text{.... Eq. 3-8}$$

where

V = velocity in ft/sec

H = depth in ft

The constants c and n can be evaluated as follows:

$$n = 1 + \frac{1}{y - 1.17} \qquad \text{.... Eq. 3-9}$$

where

y = the mean velocity increase per 5 ft increase in river stage.

The constant c is related to the slope of the stream as modified by the roughness of the stream bottom. The values of c can be determined by examination of Tables 3-2 and 3-3. The irregularity factor noted in those tables is equal to the number, per river mile, of changes in the longitudinal direction of the stream bottom exceeding a change of elevation of one foot. The exponent of H originates in the Black-Phelps diffusion formula for atmospheric reaeration.

The obvious problem with this formula is the difficulty of determining c and n. Since this original approach, a number of investigators have evolved similar formulae, giving K_a as a function of stream depth and velocity. These are as follows:

Table 3-2

Determinations of the Constant "C" in the Black-Phelps Equation
(for irregularity factor = 2.0-3.0)

c	Low Water Slope (ft/mile)
38.0	1.68
38.0	1.67
12.0	1.04
6.0	0.65
1.5	0.88
0.2	0.36
0.2	0.17

Table 3-3

Determinations of the Constant "C" in the Black-Phelps Equation
(for irregularity factor = 3.0-5.0)

c	Low Water Slope (ft/mile)
131	2.14
50	1.69
18	0.50
18	0.21

Table 3-4

A Comparison of the Constants for Various K_a Predicting Equations

Equation Number	K_a Prediction Equations Constants a	b	c
9	var.	var.	2
10	11.56	0.969	1.673
11	12.91	0.50	1.50
12	23.17	0.73	1.50
13	7.59	1.00	1.33
14	21.62	0.67	1.85

Churchill, Elmore, and Buckinghim:(9)

$$K_a = 11.56 \frac{V^{0.969}}{H^{1.673}} \qquad \text{.... Eq. 3-10}$$

O'Connor and Dobbins:(10)

$$K_a = 1.291 \frac{V^{0.5}}{H^{1.5}} \qquad \text{.... Eq. 3-11}$$

Gameson:(11)

$$K_a = 23.17 \frac{V^{0.73}}{H^{1.5}} \qquad \text{.... Eq. 3-12}$$

Langbein and Durham:(12)

$$K_a = 7.59 \frac{V}{H^{1.33}} \qquad \text{.... Eq. 3-13}$$

Owens, Edwards and Gibbs:(13)

$$K_a = 21.62 \frac{V^{0.67}}{H^{1.85}} \qquad \text{.... Eq. 3-14}$$

All these formula relate K_a to two fairly easily determined stream characteristics, depth and velocity. Unfortunately, the results that they yield are different, due to the wide variations in equation constants. If we put this equation in general form

$$K_a = a \frac{V^b}{H^c} \qquad \text{.... Eq. 3-15}$$

then the equation constants can be compared as in Table 3-4 which stresses the differences among them.

Although velocity and depth can be measured independently in the stream, these factors are both influenced by stream flow. Thus, if the relationship between flow and velocity and flow and depth are known, the reaeration constants can be presented as functions of stream flow. These relationships have been determined by Yu, for the test reach of the Passaic River as described in Appendix I-1 to Chapter I, and are:

$$H = 0.746\ Q^{0.398} \qquad \text{.... Eq. 3-16}$$

$$V = 0.020\ Q^{0.524} \qquad \text{.... Eq. 3-17}$$

Using these equations, H and V can be calculated for a given flow, and the K_a values in turn calculated from H and V. These results, for all the predicting equations except that of Black and Phelps, are presented in Table 3-5 and Figure 3-2. The reasons for these large differences are not clear, but may be due to the fact that the Passaic is an unusually slow moving stream which may be out of the range of most of the data available. It is hoped that at least part of this discrepancy will be corrected in the future, so that these equations may be used with a greater degree of confidence. At present, the equation of Owens, Edwards, and Gibbs, which incorporates the results of a number of investigators, and which in fact was the curve lying between the two curves higher and lower, was accepted as the best approximation.

The effect of temperature on K_a has been studied by a number of investigators. Churchill, Elmore, and Buckingham(8) have presented the following relationship:

$$K_a(T) = K_{a(20)} \times 1.0241^{(T-20)} \qquad \text{.... Eq. 3-18}$$

where $K_{a(T)}$ = the reaeration constant at temperature T and $K_{a(20)}$ = the reaeration constant at 20°C, the value calculated from the predicting equations. This relationship was used to adjust the basic value for temperature differences.

Net Photosynthetic Oxygen Production or P-R

Oxygen is produced by photosynthesis through the reaction of water and carbon dioxide to produce organic matter. The general reaction, according to Stumm and Morgan(14) is as follows:

Table 3-5

The Atmospheric Reaeration K_a as a Function of Stream Flow and Prediction Equation Type

Flow cfs	Velocity (ft/sec)	Depth (ft)	K_a Prediction Equation Number 10	11	12	13	14
50	0.155	3.53	0.230	0.766	0.895	0.221	0.600
100	0.222	4.66	0.205	0.605	0.766	0.219	0.458
200	0.320	6.15	0.184	0.478	0.662	0.216	0.350
300	0.395	7.22	0.173	0.419	0.612	0.216	0.299
400	0.460	8.10	0.166	0.380	0.570	0.216	0.269
500	0.517	8.85	0.159	0.352	0.543	0.216	0.246
600	0.568	9.52	0.154	0.331	0.522	0.215	0.228
700	0.616	10.12	0.150	0.315	0.506	0.215	0.216
800	0.661	10.67	0.147	0.301	0.492	0.215	0.205
900	0 703	11.18	0.143	0.290	0.478	0.215	0.196
1000	0.743	11.66	0.142	0.281	0.469	0.215	0.189

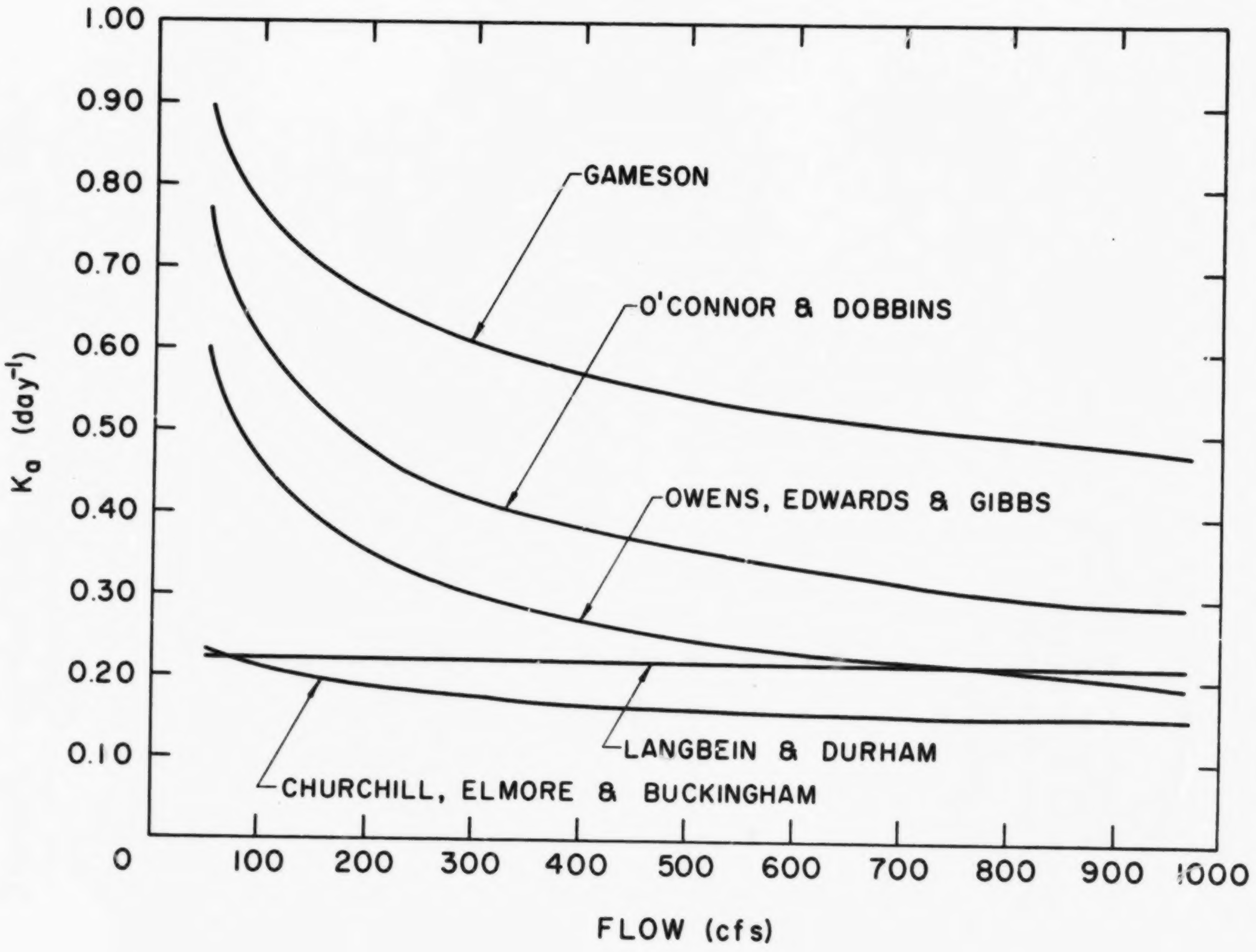

FIGURE 3-2. K_a vs. flow as calculated by various prediction equations.

$$106\ CO_2 + 90\ H_2O + 16\ NO_3^- + 1\ PO_4^{\equiv} +$$

$$\text{light energy} = C_{106}\ H_{180}\ O_{45}\ N_{16}\ P_1 + 154/2\ O_2$$

This process may be nutrient (N,P, etc.) or CO_2 limited, but in polluted water this is unlikely. Bailey(15) has expressed the contribution of photosynthesis to oxygen production as follows:

$$p = a_1\ Ch. \frac{Io^{2/3}}{K} + a_2\ T - a_3\ H \qquad \text{.... Eq. 3-19}$$

where

p = photosynthetic oxygen production in grams/m^3/day (mg/l)

a = various coefficients of multiple regression

Ch = mean chlorophyl content in grams/m^3(mg/l)

Io = mean daily solar intensity in cal/cm^2/day

K = the extinction coefficient in a water column (m).

T = mean water temperature in °C

H = mean water depth (m)

Photosynthetic oxygen is produced only during the day. At night, the reverse reaction consumes part of the oxygen so produced. In terms of the oxygen sag equation noted, photosynthetic oxygen production must be dealt with in terms of a net daily production, or a net production in multiples of a day.

In this study, the contribution of photosynthesis to the stream dissolved oxygen was evaluated using the light and dark bottle technique. In this method, pairs of bottles, consisting of one clear and one opaque, are incubated in the stream at several depths at various points in the stream. The incubation is from one to several days, depending on the rate of production of photosynthetic oxygen. At the end of the incubation period, the oxygen concentrations in the light and dark bottles are determined. This was done using the "Standard Methods" Winkler method(3) or a galvanic oxygen probe (temperature compensated and with a stirring motor) calibrated against the Winkler method in the Passaic River water. The net oxygen due to photosynthesis is equal to the difference in the oxygen concentration of the light and dark bottle taken at the end of each 24 hour period. If this is over an incubation period of several days, it is reported in terms of the average daily production.

This technique assumes that the biological processes proceed in the bottle the same way they proceed in the stream, which might not be completely true. Heukelekian and Heller(16) have criticized the method on the basis of the higher surface/volume ratio of the bottle, and Herbert(17) on the basis of the lack of turbulence in the bottle, both factors representing major deviations from stream conditions. In addition, it assumes no contribution to photosynthetic oxygen production by benthic algae or attached plants. This was actually the case for the section of the Passaic River under investigation. Whatever its deficiencies, the light-dark bottle technique is the most widely employed procedure, and so it was used in the present study.

The results of this method present the photosynthetic oxygen production in mg/l/day as a function of depth. These depth studies were done at three locations in the river. These were.

Station 1: 700 ft. from mechanical aerator

Station 2: 1100 ft. from mechanical aerator

Station 3: 1800 ft. from mechanical aerator

Each station was in the main stream channel in unshaded areas. The results of this study are found in Tables 3-6 and 3-7.

To interpret this data, a linear relationship between depth and photosynthetic oxygen production was assumed, and this is substantiated by data presented in

Table 3-6

(P-R) as Determined by Light and Dark Bottle Studies
Upper Passaic River, August 1968

Date	Depth (inches)	Station 1 mg/l	Station 2 mg/l	Station 3 mg/l	Average at 4" depth mg/l
8/25/68	4	10.5	10.3	11.3	10.7
	14	3.5	-	-	
	17	2.4	-	-	
	22	0.0	-	-	
8/27/68	4	2.1	2.0	2.2	2.1
	12	1.0	-	-	
	15	0.0	-	-	
	20	0.0	-	-	
8/28/68	4	5.4	5.1	5.2	5.2
	14	2.9	-	-	
	16	1.5	-	-	
	19	1.0	-	-	
undated*	4	4.0	3.9	3.7	3.9
	4	5.5	5.6	5.0	5.4
	4	2.6	2.3	2.6	2.5
	4	8.2	7.8	7.1	7.7
	4	7.1	6.8	7.0	7.0
	4	6.5	6.3	6.7	6.5
	4	5.6	5.5	5.8	5.6
	4	7.1	7.6	6.6	7.1
	4	8.3	8.4	8.1	8.3

---

* Data originally intended for a light intensity correlation, so no date was recorded. When intensity meter failed the data was included here.

Table 3-7

(P-R) as Determined by Light and Dark Bottle Studies
Upper Passaic River, September 1968

Date	Depth (inches)	Station 1 mg/l	Station 2 mg/l	Station 3 mg/l	Average at 4" depth mg/l
9/15/68	4	2.4	3.1	-	2.7
	6	0.4	-	-	
	12	0.0	0.0	-	
	15	0.0	-	-	
9/11/68	12	0.5	0.5	-	0.5
	22	0.0	0.0	-	
9/13/68	6	0.9	0.8	-	0.8
	14	0.2	0.4	-	
9/16/68	6	2.8	3.0	-	2.9
	14	0.8	1.1	-	
9/18/68	6	1.1	1.3	-	1.2
	14	0.2	0.1	-	
	20	0.1	-	-	
9/20/68	6	1.4	1.5	1.4	1.4
	14	0.8	0.2	-	
	20	0.0	0.0	-	
9/23/68	6	4.4	4.2	4.6	4.4
	14	1.8	1.7	-	
	20	0.1	-	-	
9/27/68	6	5.8	6.2	-	6.0
	14	3.7	3.9	-	
	20	2.8	-	-	

Figure 3-3. Using this linear relationship, it was found that the eutrophic (net oxygen producing) zone was limited to the upper 50 cm of stream water, and that the average net photosynthetic oxygen production was 2.3 mg/1 (based on a "near surface" average value of 4.6 mg/1).

As net photosynthetic oxygen production takes place only in the upper 50 cm of the river, stream flow will effect this value through its effect on depth. The increased depths that occur with increased flow will "dilute" out this photosynthetic effect. Again using equation 3-16, $H = 0.746\ Q^{0.398}$, it is possible to calculate the photosynthetic oxygen contributions as a function of flow (depth). For example, at a flow equivalent to a depth of 2 meters, the contribution would be $50/200 \times 2.3 = 0.6$ mg/l. this data is presented in Table 3-8 and Figure 3-4. This represents the average only. Obviously, on any given day, this effect can vary greatly. As this does represent a summer average, no temperature correction factor is included here, although P-R is temperature dependent.

The dissolved oxygen level in a water body in which substantial photosynthesis is occurring undergoes what is called "diurnal" fluctuations. Thus the dissolved oxygen level reaches a maximum value during the day when photosynthesis is at a maximum, and a minimum at night when photosynthesis is at a minimum. An example of this for the Passaic River is found in Figure 3-5. This also substantiates the photosynthesis observed by the light-dark bottle technique.

The Deoxygenation Constant K_d and the Ultimate Oxygen Demand L_o

The fate of organic matter entering a stream depends on its physical nature. Soluble and fine particulate matter is oxidized in the stream itself by the microbial population present. The larger particulates may settle and form deposits in the bottom of the stream. Here too, the organics undergo decomposition, but due to the place and the rate at which this oxygen demand is exerted, this latter is considered a separate process.

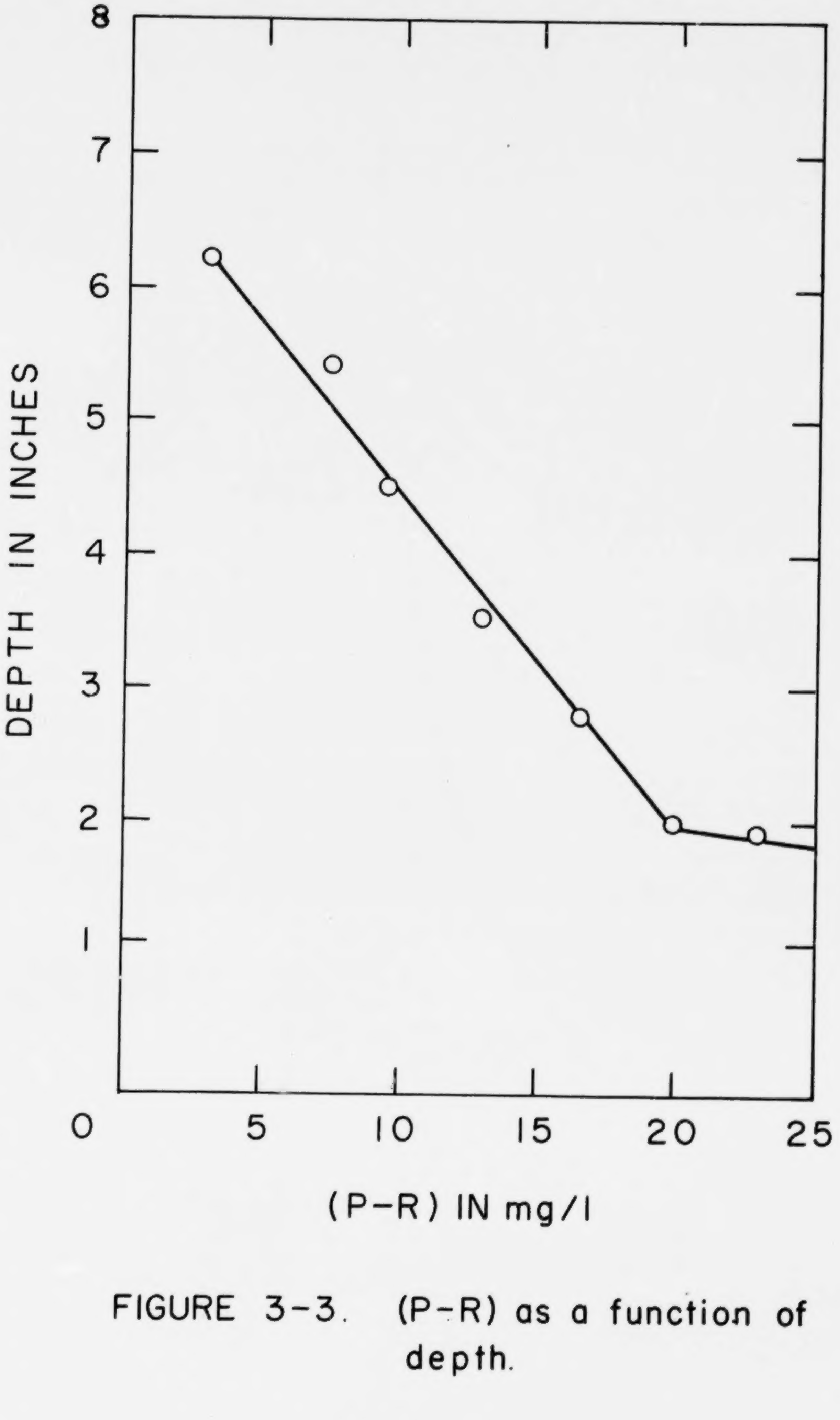

FIGURE 3-3. (P-R) as a function of depth.

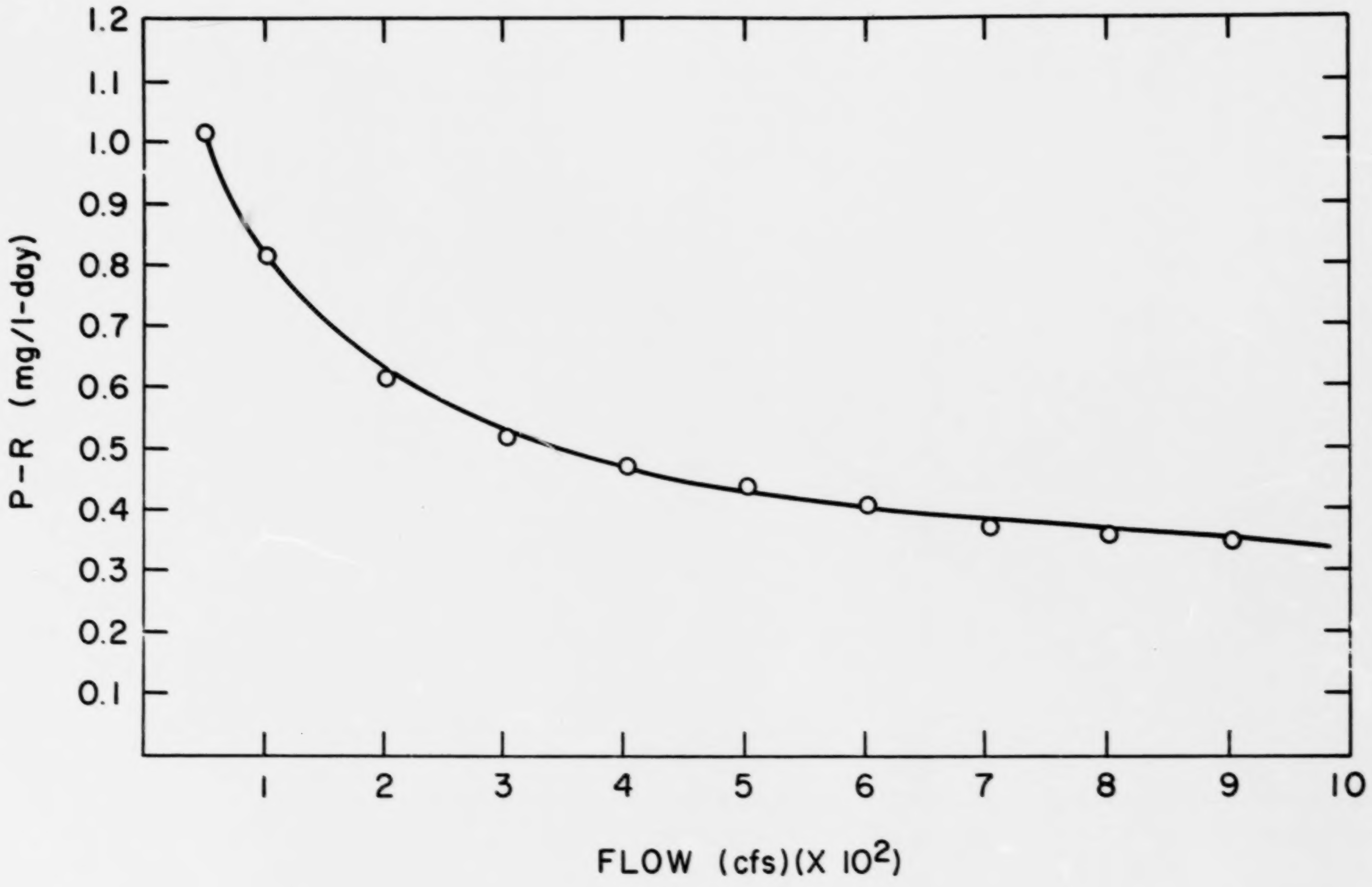

FIGURE 3-4. P-R as a function of flow.

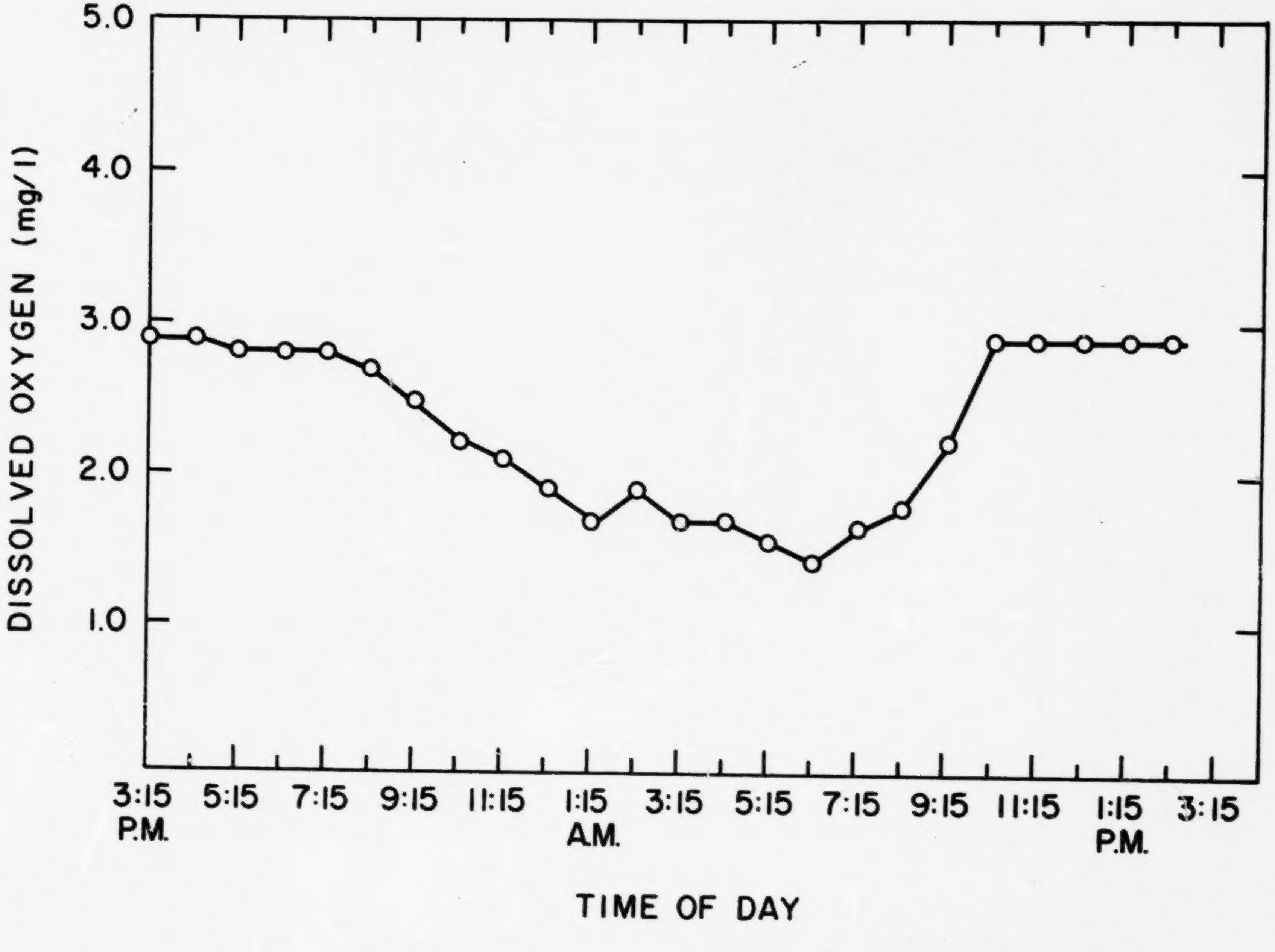

FIGURE 3-5. Diurnal fluctuation in dissolved oxygen (Passaic River, Sept. 2-3, 1968).

Table 3-8

Net Photosynthetic Oxygen as a Function of Flow
Passaic River at Pine Brook, Summer Conditions

Flow (cfs)	Depth (ft)	P-R mg/l/d
50	3.53	1.07
100	4.66	0.81
200	6.15	0.61
300	7.22	0.52
400	8.10	0.47
500	8.85	0.43
600	9.52	0.40
700	10.12	0.37
800	10.67	0.36
900	11.18	0.34
1000	11.66	0.32

In the oxygen sag equation presented, the oxidation of organics in the stream is considered a first order process, even though it has been fairly well established that it does not entirely correspond to first order kinetics.(18)(19)(20)(21) The equation relating the oxygen demand of this organic matter (BOD) with time is

$$Y_\tau = L_o(1-e^{-K_d\tau}) \qquad \text{.... Eq. 3-20}$$

where Y_τ = the oxygen utilized at time τ and the other constants are as previously defined.

There are several ways of estimating the deoxygenation constant K_d. One method is to set up sampling locations on the stream a known distance apart. The five day, 20°C BOD is then determined at each location on the same stream water segment. If the velocity is known, τ can be calculated, and a plot of log BOD vs τ can be prepared. This plot will have a slope of $-K_r$ where $K_r = 2.303\ k_r$. The constant K_r is used here as this plot gives the rate constant of BOD removal from the stream. Frequently, a curve of the type given in Figure 3-6 will be observed. This plot can be used to determine both K_r and K_d. If only one line is observed, the slope may be assumed to represent K_d, also but this might not be correct.

Another method is to study the oxygen uptake of stream samples at 20°C in the laboratory as a function of time. A table of Y_t vs t can be prepared, and L_o and K_d can be calculated. Assuming that t and τ are equivalent, these results can be "projected" to the stream. Using this method, L_o and the deoxygenation constant K_d can be calculated.

The results presented in this section were based on the projection method.* The oxygen uptakes of stream water samples at 20°C were observed as a function of time using a Warburg Respirometer. Nitrification was inhibited using allylthiourea.(21) A typical curve is presented in Figure 3-7. All samples were obtained at the aeration site. Unfortunately, equation 3-20 cannot be solved directly for K_d and L_o from y vs τ data. Because of this,

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*As noted later in Chapters VI and VII, K_r values measured by field data also were available for comparison.

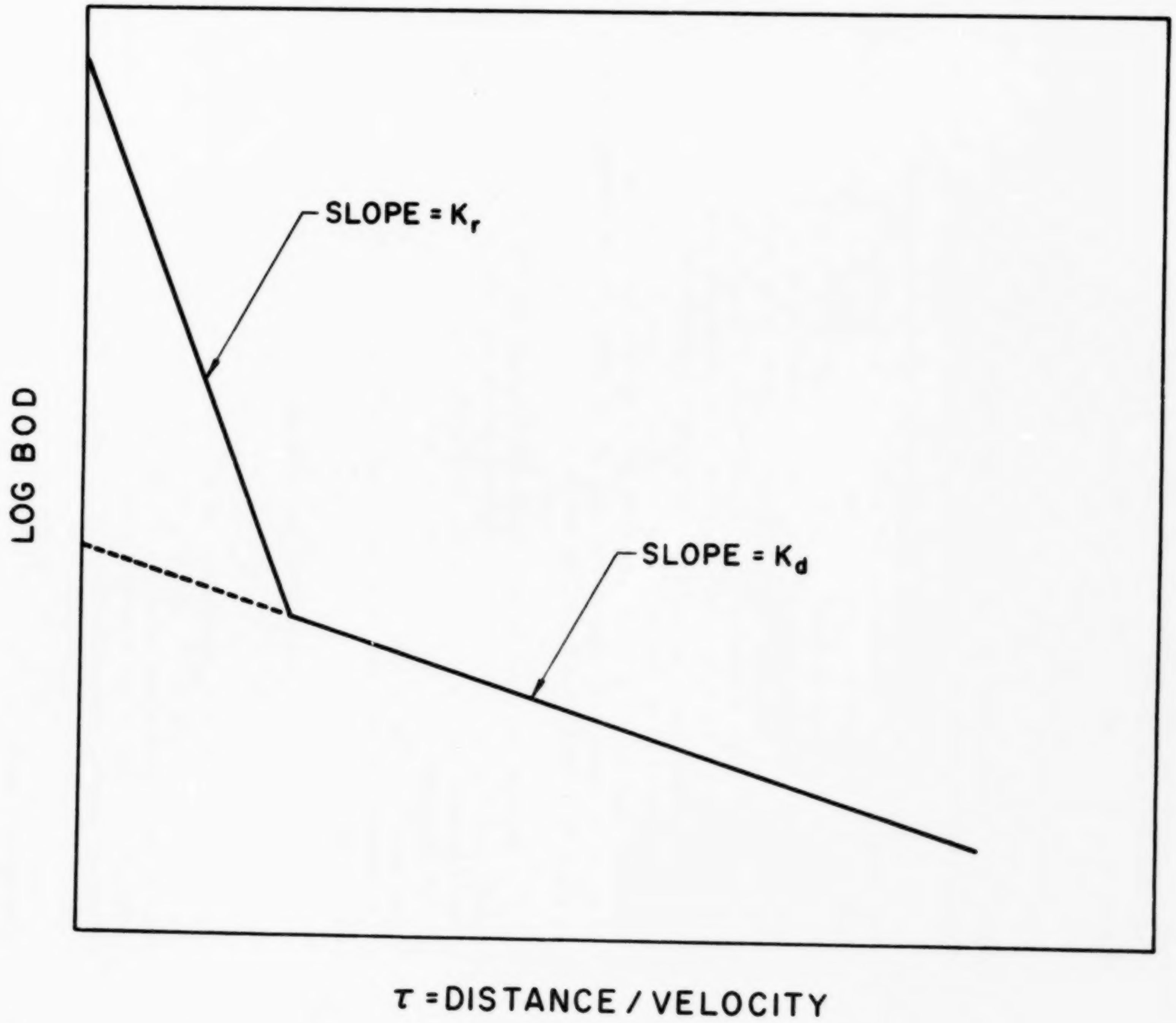

FIGURE 3-6. Generalized curve of BOD vs. travel time.

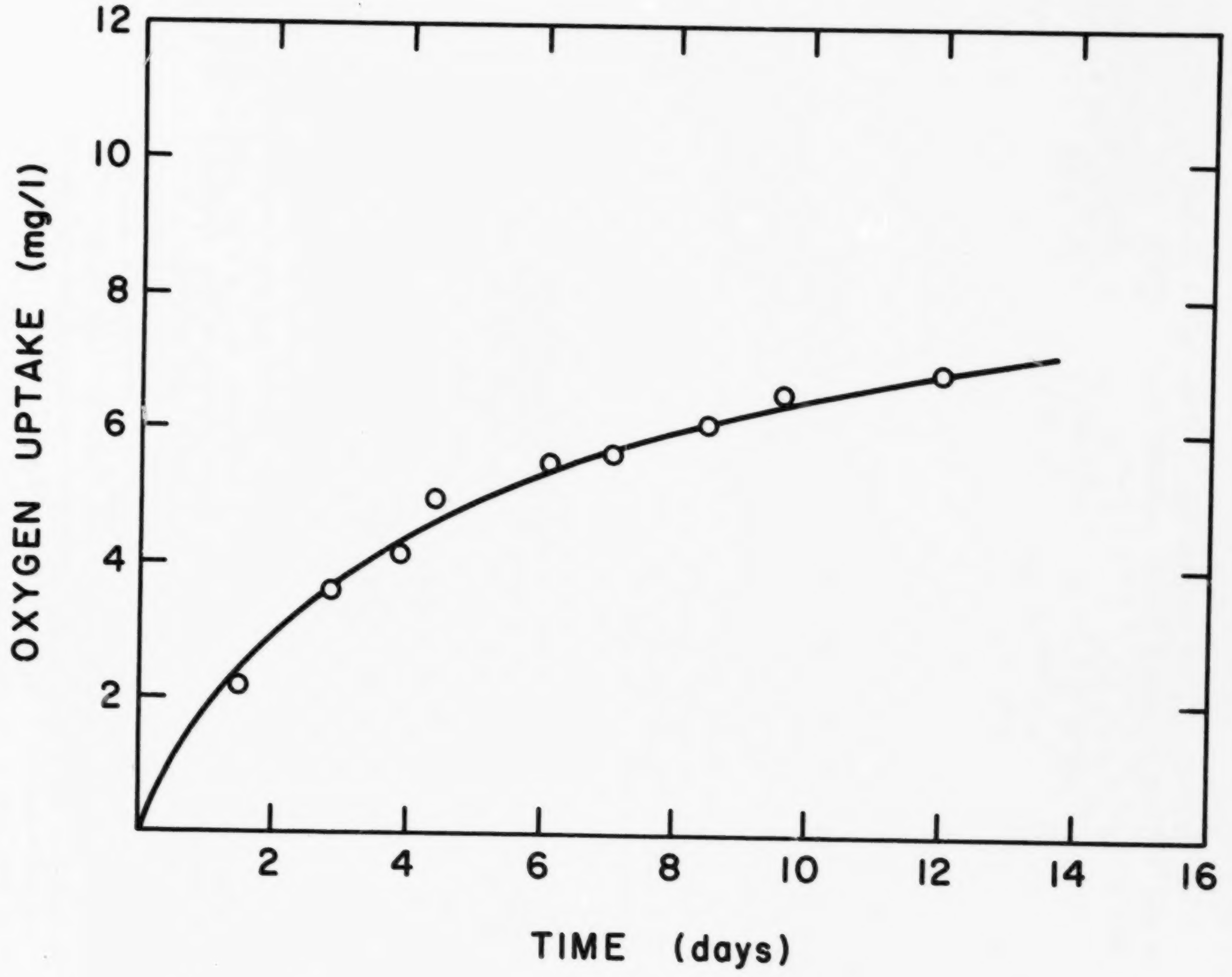

FIGURE 3-7. Carbonaceous BOD as a function of time.

there have been a number of methods published for the estimation of the equation constants from oxygen uptakes vs time data. (23)(24)(25)(26)(27)(28)(29)

For reasons of simplicity, the two point method of Rhame(30) was employed in the present study. Using this method:

$$L_o = \frac{X^2}{2X-Z} \qquad \text{.... Eq. 3-21}$$

and

$$K_d = \frac{2.3}{T-t} \log \frac{X}{Z-X} \qquad \text{.... Eq. 3-22}$$

where

X = the oxygen uptake at time t

Z = the oxygen uptake at time T

T = 2t

Two sets of K_d and L_o values were calculated from the data. These were where t=2 and T=4 days, and t=5 and T=10 days. the constants are different over these periods due to the fact that the oxidation does not exactly follow first order kinetics, and therefore the equation parameters are not independent of the time used in their estimation. These results are presented in Table 3-9. The average rate constant K_d was 0.356 days-1 and the average ultimate oxygen demand L_o was 7.7 mg/1 for a four-day oxidation period.

The role of flow in this instance is by no means as clear as it is in the calculation of K_a or the dilution effect for photosynthetic reoxygenation. An increase of flow might as a first approximation be assumed to decrease the L_o concentrations due to dilution effects. However, the data in Figure 3-8 indicate that a five-fold increase in flow (200 to 1000 cfs) decreases the five-day 20°C BOD concentration less than two fold (6.1 to 3.6 mg/1). Other data over a period of years substantiate that in addition to the dilution effect there is substantial increases in the total BOD load with increased discharge. This may be due partly to resuspension of stream organic deposits and partly organics entering the stream in the runoff that is the cause of the stream flow increases.

Table 3-9

Ultimate Oxygen Demands and Rate Constants for Passaic River Water
(all results at 20°C)

Sample Type	Date	4 day Period		10 day Period	
		K_d days^{-1}	L_o mg/l	K_d days^{-1}	L_o mg/l
Composite	7/26/68	0.323	8.8	0.288	9.4
Composite	8/19/68	0.581	6.9	0.451	7.5
Composite	8/22/68	0.307	8.1	0.226	9.6
Composite	8/27/68	0.346	6.0	0.244	7.2
Composite	8/28/68	0.398	10.9	0.206	15.2
Grab, 9:15AM	9/3/68	0.279	6.5	0.219	7.2
Grab, 3:15AM	9/3/68	0.202	7.2	--	--
Grab, 9:15AM	9/3/68	0.373	6.5	0.290	8.2
Grab, 3:15AM	9/4/68	0.405	6.5	0.194	9.4
Grab, 9:15AM	9/4/68	0.346	9.2	0.306	9.5
Average		0.356	7.7	0.269	9.2

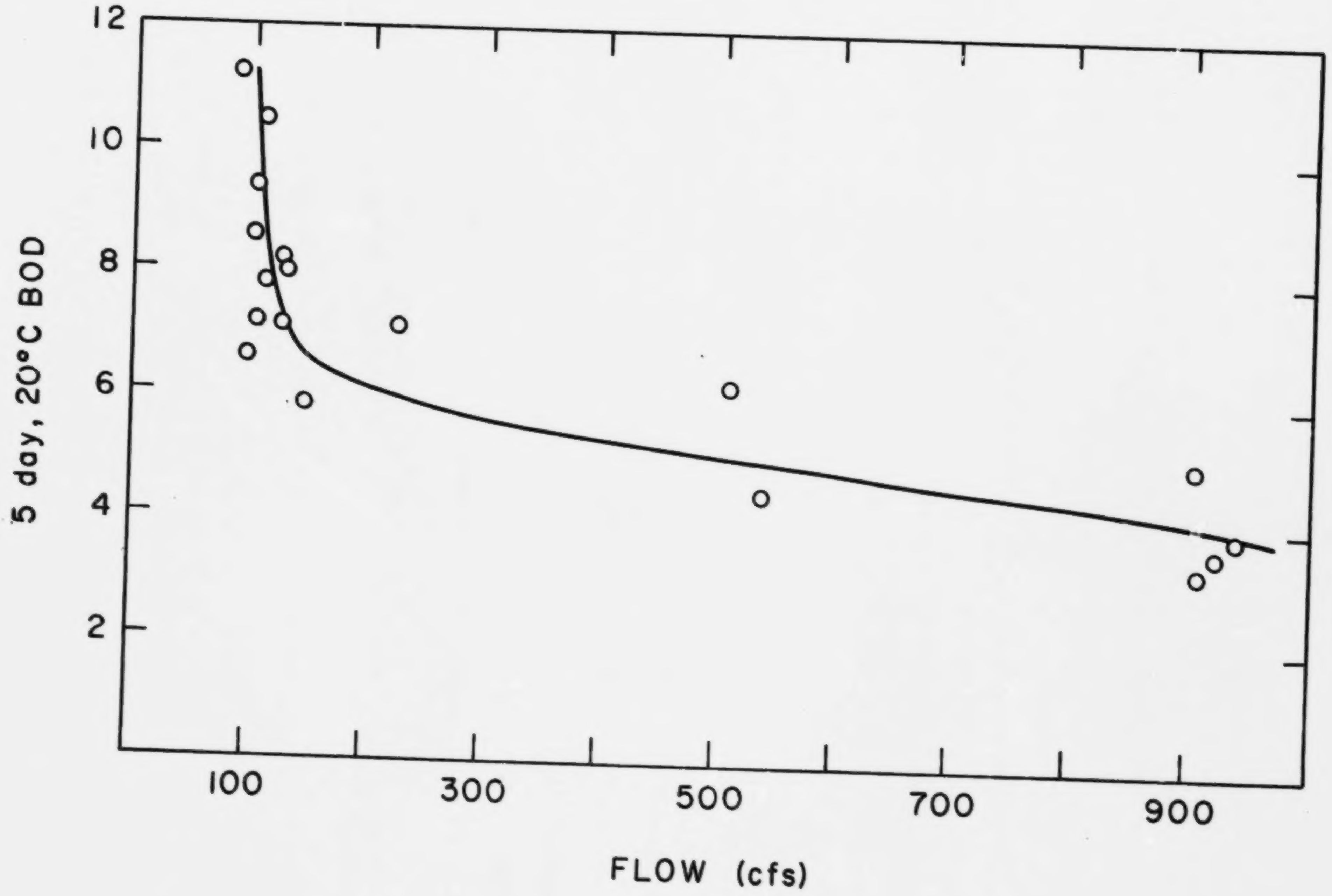

FIGURE 3-8. 5 day, 20°C BOD as a function of flow.

There have been many relationships evolved for the dependence of K_d and L_o on temperature. A recent study by Zanoni(31) gave the following relationships.

$L_o\ (T) = L_{o(20)}\ (1 + 0.0113\ (T-20)).$ .... Eq. 3-23
range 20° - 35°C

$L_o\ (T) = L_{o(20)}\ (1 + 0.0033\ (T-20)).$ .... Eq. 3-24
range 2° - 20°C

$K_d\ (T) = K_{d(20)}\ (0.796)\ (1.126^{T-15}).$ .... Eq. 3-25
range 2° - 25°C

$K_d\ (t) = K_{d(20)}\ (1.047^{T-20})$ range 15° - 32°C .... Eq. 3-26

$K_d\ (T) = K_{d(20)}\ (1.728)\ (0.985^{T-32})$
range 32° - 40°C .... Eq. 3-27

The Benthal Oxygen Demand "S"

When particulate organic matter settles to the bottom of a stream the processes of decomposition continue. Oxygen diffuses from the stream into the bottom deposits, where it is used by the microbial population of the deposit to oxidize the organic constituents. In addition, anaerobic fermentation processes may result in the conversion of particulate organics to soluble organics, methane, and CO_2. Anaerobic conditions also may cause reduction of sulfate to hydrogen sulfide, which can exert an oxygen demand in the bulk of the stream.

Compared to the rate of oxidation in the stream proper, these benthal processes are quite slow. Nevertheless, they could exert a significant demand on the oxygen resources of a stream.(32) This demand does not take place in the bulk of the stream but at the stream-mud interface. It is therefore considered an areal demand, and is reported in terms of weight (grams) of oxygen consumed/unit area of bottom (m^2).

There have been approaches to estimating this demand from deposit characteristics and other factors.

For example, Fair, Moore and Thomas(33) in their classic work on benthal decomposition, evolved the following relationship:

$$\gamma_m = 3.14 \cdot 10^{-2} \gamma_o C_T W \cdot \frac{5+160\ W}{k-160\ W} \cdot (t_a)^{1/2} \qquad \text{.... Eq. 3-28}$$

where

γ_m = maximum daily benthic oxygen demand in grams/square meter

γ_o = the five day 20°C BOD in grams per Killogram of sludge volatile matter

C_T = temperature factor, or y_c/y_o where y_t is y_o at deposit temperature. This can be calculated from BOD - temperature relationships previously described.

W = daily rate of deposition of volatile solids in killograms/square meter

t_a = time in days, up to 365, during which accumulation takes place

The difficulties involved in determining the equation parameters limits it use, and the oxygen uptakes are usually measured directly.

The most common method of evaluating benthic oxygen demand is to obtain a sample of a stream benthal deposit, and measure the oxygen uptake of this deposit in the laboratory. Due to the disturbance of the deposit when using this technique, it is difficult to state with any degree of assurance that these laboratory values are representative of the actual oxygen consumption values of the undisturbed deposits in the stream bottom.

A better way to evaluate benthal oxygen demands is to measure the oxygen consumption of the deposit "in situ." Such field respirometers have been described by O'Connell and Weeks(34) and by Stein and Denison(35). The field benthal respirometer used in this study was designed after Stein and Denison, and is described by Figure 3-9. This respirometer is an opaque chamber that is lowered to the stream bottom where its only open face is in contact with the mud surface. The oxygen level of the water in the chamber is then determined for as long a period of time as possible. The oxygen level is again measured at the end of the contact period, which is signified by the oxygen level dropping below 1 mg/l.

The oxygen consumed is calculated from the difference in oxygen concentration at the start and end of the run. This value is corrected using a BOD run at the same temperature and time. From a knowledge of the contact area, volume, and time, the benthal demand may be determined. The method of calculation is

$$\text{Benthal Demand} = \text{Oxygen consumed} - \text{BOD} \times \frac{1}{\text{area}} \times \frac{1}{\text{time}} \times \text{Volume}$$

where oxygen consumed and BOD are in mg/l, volume is in liters, area in square meters, time in days, and benthal demand in mg/meter^2/day. Due to low initial dissolved oxygen concentrations, the times over which demands were measured were usually only several hours in length. This demand was projected in a linear fashion, although the uptakes were not quite linear, as is shown by Figure 3-10. The calculations and results of the studies run in the Passaic River are presented in Table 3-10. The sampling locations are presented in Table 3-11.

The average value of the benthal oxygen demands was 2220 mg/meter^2/day, with a range of 1350 to 3790 mg/meter^2/day. This variation may reflect differences in deposit depth, availability of deposit organics, concentration of organics, etc.

A study was also made to determine the fraction of the bottom covered by mud deposits. It was estimated from the data presented in Table 3-12 that 22% of the bottom was covered by mud. Therefore, the demand on the river by the deposits was 0.22 x 2220 or 488.6 mg/m^2/d.

In the oxygen sag equation, the benthal demand "S" is in terms of mg/l/day and not as the areal demand of

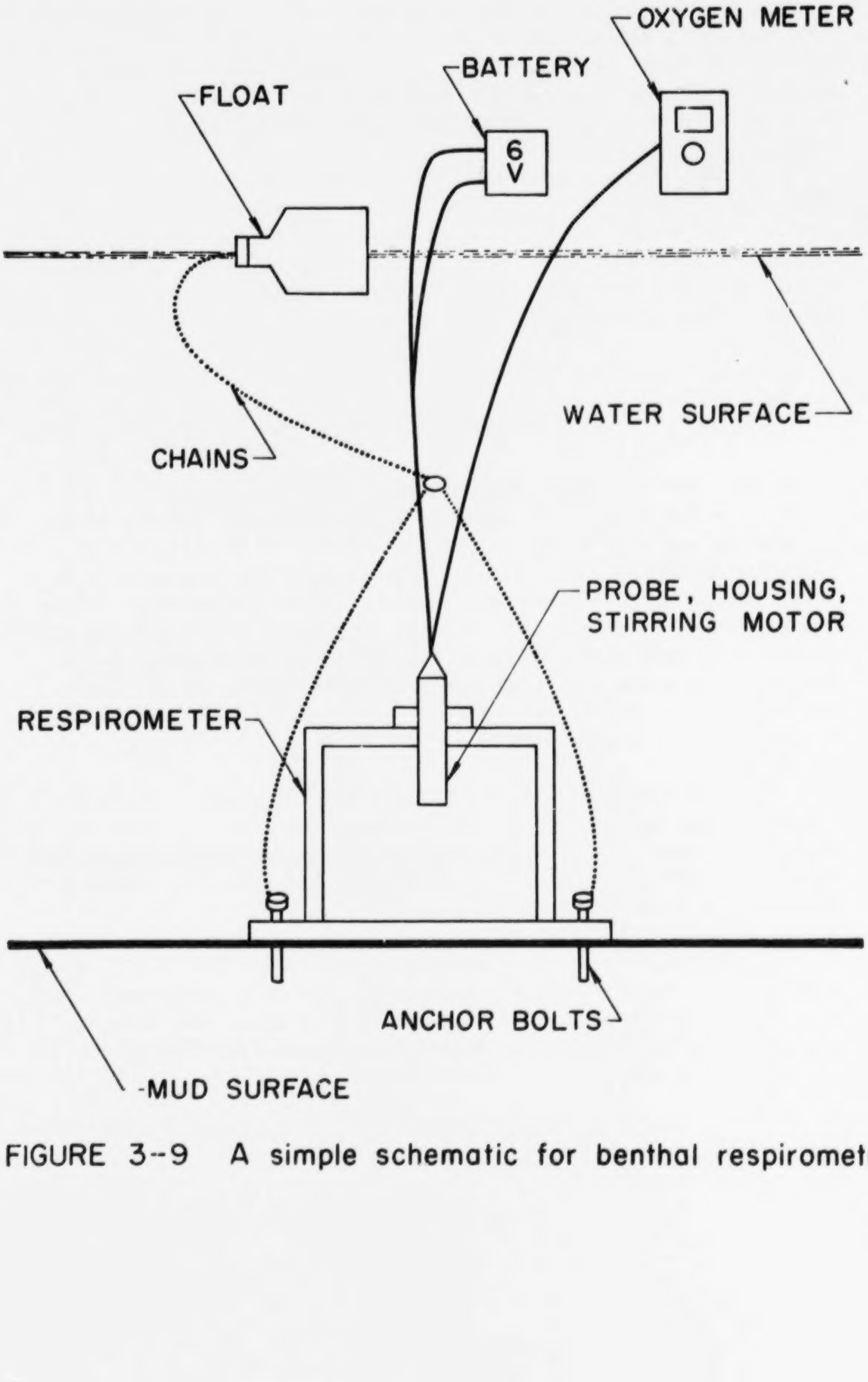
OXYGEN METER
BATTERY
FLOAT
6 V
WATER SURFACE
CHAINS
PROBE, HOUSING, STIRRING MOTOR
RESPIROMETER
ANCHOR BOLTS
MUD SURFACE

FIGURE 3-9 A simple schematic for benthal respirometry.

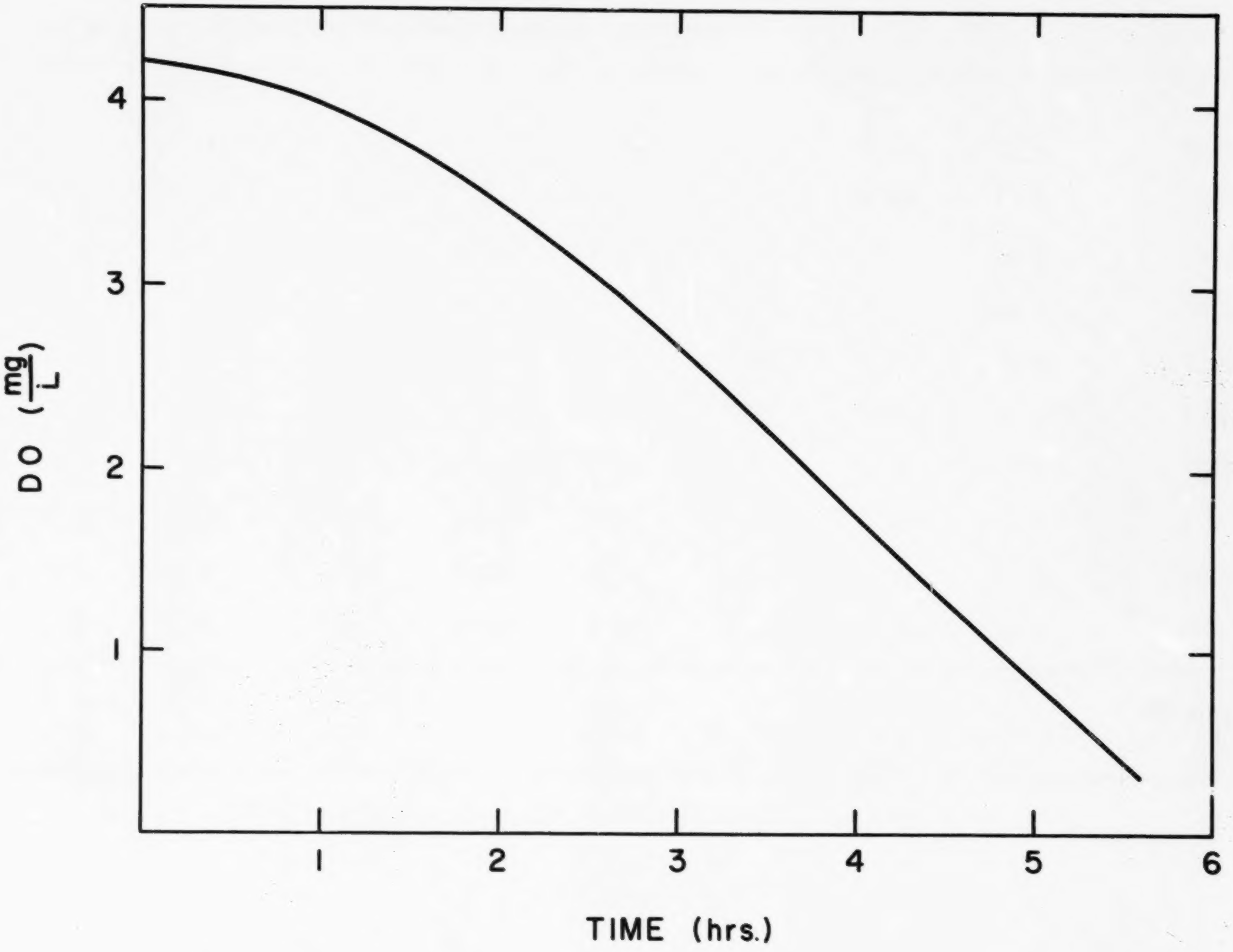

FIGURE 3-10. Respirometer DO as function of time.

Table 3-10

Benthic Oxygen Demand as Calculated from Respirometer DO Values and BOD

(A) Run #	(B) D.O. initial mg/l	(C) D.O. Final mg/l	(D) time hrs	(E) time days	(F) BOD mg/l	(G) B - C	(H) G - F	(I) H x 28	(J) I/E x 0.0994 $mg/d/m^2$
1 *	6.0	2.35	15	0.625	2.34	3.63	1.31	36.7	--
2	3.0	0.30	5.5	0.229	0.70	2.70	2.00	56.0	2460
3 *	1.78	0.78	3.25	0.136	0.50	1.00	0.50	14.0	--
4	1.95	0.50	2	0.083	0.60	1.45	0.85	23.8	2870
5	1.50	0.05	2	0.083	1.00	1.45	0.45	12.6	1520
6	2.85	1.05	3	0.125	1.30	1.80	0.50	14.0	1130
7	1.40	0.30	1.5	0.063	0.40	1.10	0.70	19.7	3180
8	4.30	1.00	5.	0.208	0.50	3.30	2.80	78.5	3790
9	2.90	1.50	4.23	0.177	0.55	1.40	0.85	23.8	1354
10	1.15	0.25	3	0.125	0.25	0.90	0.65	18.2	1470

* A good seal was not achieved between respirometer and mud.

Table 3-11

Location of the Sample Sites

Run	Date	Distance from Aerator (ft.)	Temperature °C Initial	Final
1	7/20/68	1,100	24.0	22.3
2	7/29/68	3,000	22.4	24.5
3	8/5/68	7,000	23.4	24.3
4	8/14/68	13,000	21.5	22.1
5	8/16/68	14,000	21.0	21.8
6	8/20/68	15,000	21.6	22.6
7	8/22/68	3,300	22.8	23.7
8	8/28/68	4,000	22.1	23.8
9	9/3/68	6,500	20.2	23.9
10	9/5/68	8,250	21.0	21.7

TABLE 3-12

EXTENT OF BENTHAL DEPOSITS

Distance from Aerator (ft.)	Distance Across Stream (ft.)	Distance covered by Mud (ft.)
500	74	30
1,000	75	13
1,500	73	0
2,000	80	0
2,500	60	10
3,000	80	17
3,500	120	120
4,000	75	25
4,500	62	12
5,000	45	0
5,500	70	10
6,000	60	0
6,500	53	3
7,000	80	20
7,500	53	3
8,000	70	14
8,500	48	8
9,000	85	17
9,500	86	26
10,000	60	8
10,500	73	10
11,000	66	22
11,500	70	22
12,000	81	28
12,500	84	6
13,000	75	3
13,500	90	15
14,000	108	15
14,500	92	18
15,000	85	15
15,500	75	22
16,000	80	15
	Avg. = 74.63	Avg. = 16.45

Fraction = 0.22

To bring these results to a concentration basis, it was necessary to calculate the volume over a square meter of river bottom. As grams/m^3 are equivalent to mg/l, the demand in mg/m^2/d is divided by 1000 and also divided by the depth in meters.

The depth in meters was in turn determined from the flow using the predicting equation of Yu (App. I-1), which was

$$H = 0.746\ Q^{0.398} \qquad \text{.... Eq. 3-29}$$

where

Q = stream flow in cfs

H = stream depth in feet

The H values in feet were converted to meters, and the benthal oxygen demand "S" calculated as a function of stream flow. These results are presented in Table 3-13 and Figure 3-11.

This is an average demand determined over the summer, and therefore, for summer conditions it should not be necessary to correct results for temperature, although such biological uptakes are undoubtedly temperature dependent.

Nitrification

The oxygen sinks previously noted were due to the oxidation of organic matter either in the stream itself or in deposits at the stream bottom. There is an addition oxygen sink caused by the oxidation of ammonia and nitrites. The overall equation for ammonia oxidation is:

$$NH_3 + 2O_2 = HNO_3 + H_2O$$

This reaction states that 1 part of ammonia could use up to 3.8 parts of oxygen. Consideration of the fact that even complete (secondary) waste water treatment plant effluents contain considerable amounts of ammonia, this can represent a significant demand on the oxygen economy of the stream.

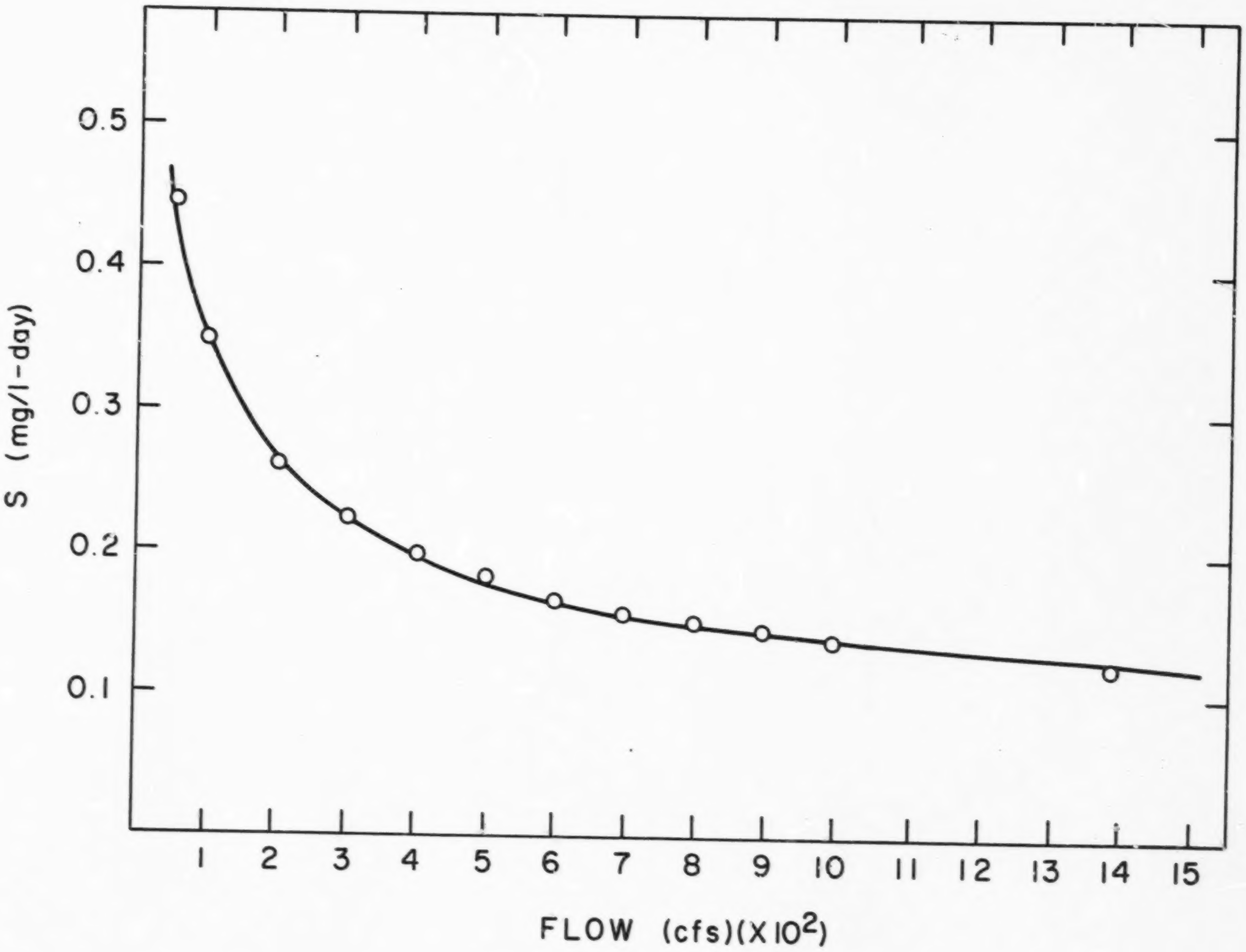

FIGURE 3-11. Benthal oxygen demand as a function of stream flow.

TABLE 3-13

BENTHAL OXYGEN DEMAND AS A FUNCTION OF FLOW

Flow cfs	Depth ft.	Benthal Oxygen Demand mg/l/day
50	3.53	0.440
100	4.66	0.344
200	6.15	0.261
300	7.22	0.222
400	8.10	0.198
500	8.85	0.181
600	9.52	0.168
700	10.12	0.157
800	10.67	0.151
900	11.18	0.144
1000	11.66	0.120
1400	12.12	0.120

Unlike the heterotrophic oxidation of organic matter, which is carried out by a very large number of different organisms, nitrification is an autotrophic process carried out by relatively few organisms. The first part of the oxidation is carried out by organisms of the genus nitrosomonas as follows:

$$2\ NH_3 + 3O_2 = 2HNO_2 = 2H_2O$$

The second part of the oxidation is carried out by organisms of the genus nitrobacter as follows:

$$2NO_2 + O_2 = 2HNO_3$$

As members of both groups are autotrophes, they oxidize ammonia and nitrite for energy, and use inorganic carbon water and minerals to form organic cell constitutents.

Nitrification is much more influenced by such factors as pH, temperature, and dissolved oxygen than is carbonaceous oxidation. An even greater limitation on nitrification is that relatively few numbers of nitrifying organisms are found in the natural environment. This, as well as the fact that the generation time can be as long as 31 hours(36), means that there is an appreciable lag between the time that ammonia is introduced to the stream, and the time when measurable nitrification occurs. Thus, until a sufficiently large population is developed, the rate of nitrification is quite slow. After a nitrifying population has built up, nitrification may proceed at a high rate until it is once more limited due to the depletion of ammonia. Thus, a nitrification curve, that is, a plot of oxygen uptake (by ammonia oxidation) vs time, frequently resembles a bacterial growth curve.

Unfortunately, the mathematics of this curve do not permit its inclusion in the oxygen sag curve previously presented. For this reason, nitrification is frequently assumed to be a first order reaction, if it were a first order process, the equation depicting nitrification would be

$$\frac{dN}{dt} = K_N\ (No - N_t) \qquad \text{.... Eq. 3-30}$$

which integrates to

$$N_t = N_o \ (1 - e^{-K_N t}) \qquad \text{.... Eq. 3-31}$$

where

N_t = nitrification demand at time t in mg/1

N_o = the ultimate nitrification demand in mg/1

K_N = the nitrification rate constant in reciprocal time units

Unfortunately, in general nitrification is not represented by the first order equation, for reasons already mentioned. This was the general case for the Passaic River studies. In this case, the BOD lag formula of Thomas(36) offers some hope. In its integral form, this equation is:

$$N_t = N_o \ (1 - e^{-K_N \ t - t_a}) \qquad \text{.... Eq. 3-32}$$

where all the terms are as previously described except t_a, which is the lag time.

The value of the constants for the description of the course of nitrification can be obtained in several ways. Indirectly, it can be done through a measurement of the production of nitrates in the stream. This must be done under steady state conditions or on the same segment of stream water as it proceeds downstream. The measured concentrations of nitrate can be converted into the equivalent amount of oxygen required to produce it through oxidation of ammonia, and a plot of oxygen consumed by nitrification vs time (in stream) can be prepared.

Another method is to obtain a sample of stream water and observe its oxygen uptake as a function of time with and without a nitrification inhibitor added. Examples of these curves are found in Figures 3-7 and 3-12. If the inhibited curve (organic matter oxidation only) is subtracted from the unhibited curve (organic matter and ammonia oxidation), a curve of oxygen utilized by ammonia oxidation will result. This curve, obtained by difference, is presented in Figure 3-13.

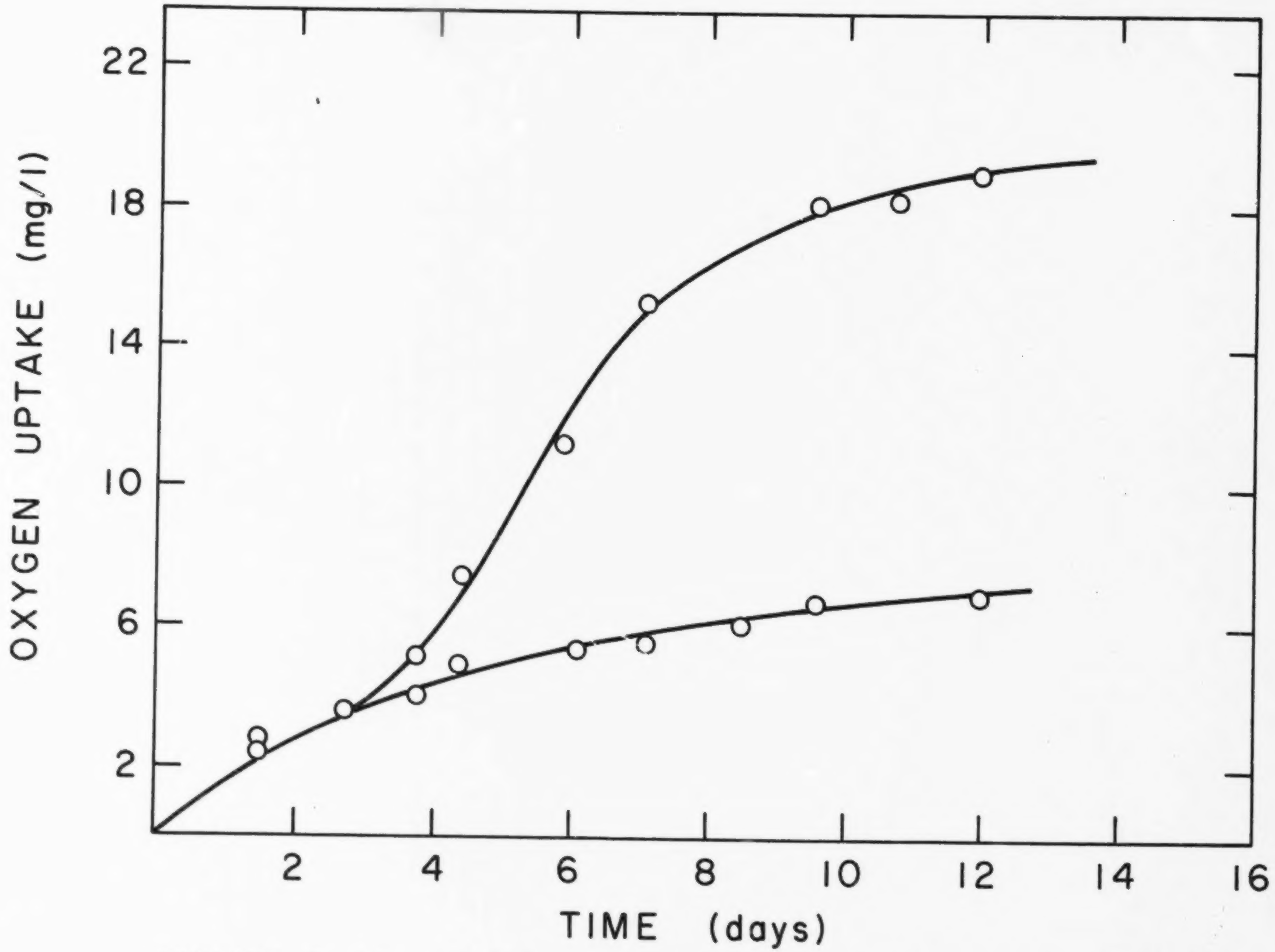

FIGURE 3-12. Total carbonaceous and nitrogenous BOD as a function of time.

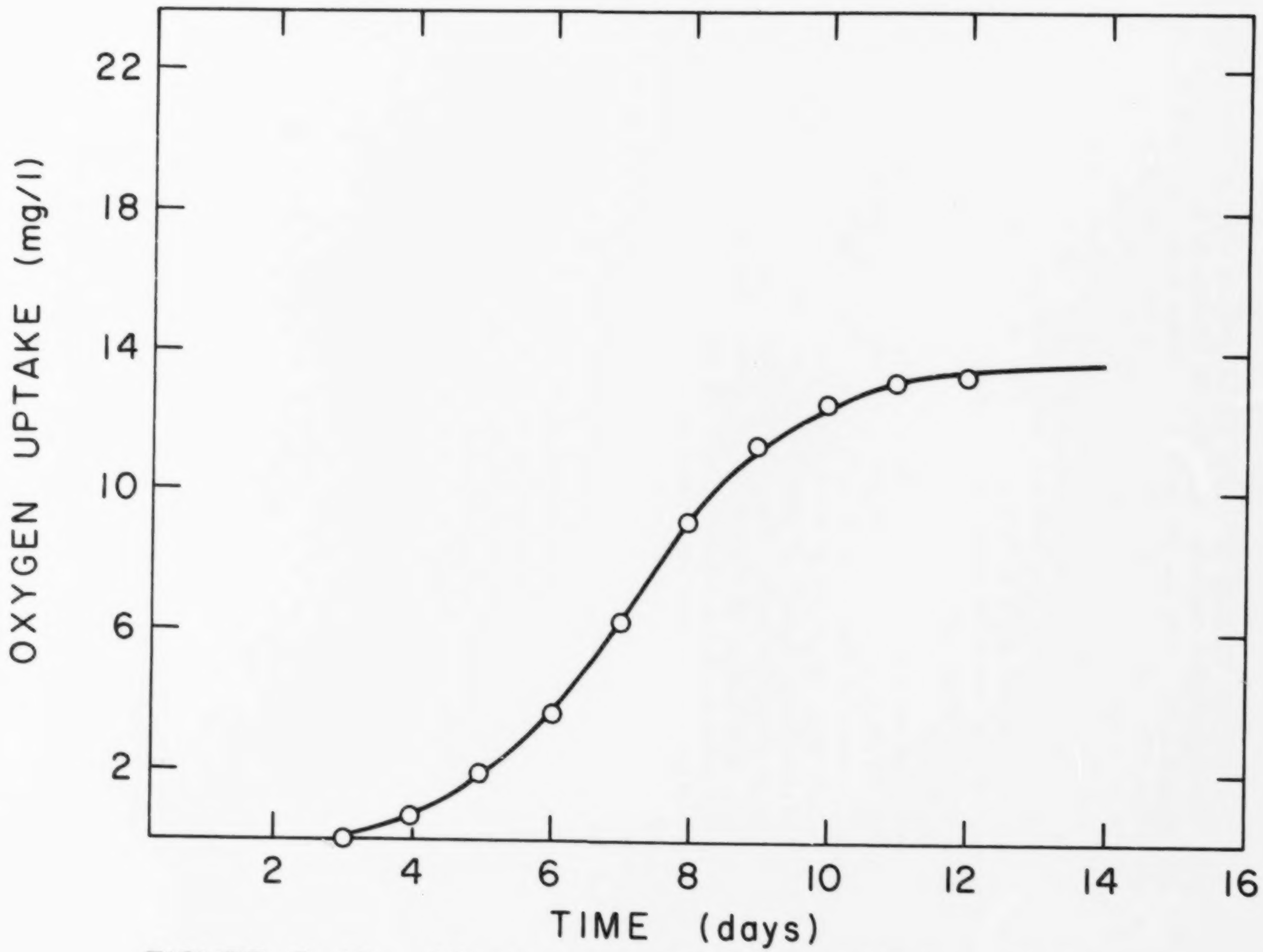

FIGURE 3-13. Nitrogenous BOD as a funtion of time.

The first method is limited by the fact that nitrates can be utilized by algae after production, and that the analytical methods for nitrate determinations are rather poor. The second method is limited by the fact that the temperature (20°C) of the laboratory oxidation as measured by a Warburg Respirometer is lower than summer stream temperatures, and the dissolved oxygen level under laboratory conditions is higher. In the case of nitrification, it is difficult to assess the effect of these changes on the lag time and rate. The second procedure was used, however, as it gave L_o and K_d data as well as nitrification data.

The determination of the lag time was arbitrary, but was performed the same way in each case, by continuing flat part of the slope to the time axis. The N_o and K_n values were obtained from the data over a six-day period using the method of Rhame(30), and the results are presented in Table 3-14. The rate constants were, on the average, similar to carbonaceous oxidation, but in the case of nitrification, the average ultimate oxygen demands were almost double that of carbonaceous oxidation, having a mean of 16.3 mg/l and a range of 13.7 to 21.7 in seven samples. The lag times were rather long, being three days on the average, also with considerable variation.

Due to the method of determinations, only the ultimate nitrifications demand can be assumed to be reasonably reliable. The values obtained for lag time and rate constants may have been considerably influenced by the differences between field and laboratory conditions. However, nitrifications does occur over this reach of stream(37). There was insufficient data to observe the role of flow, but according to O'Connor(3), the temperature relationship is

$$K_n (T) = K_n (20)\ 1.09^{(T-20)} \qquad \text{.... Eq. 3-33}$$

Summary

The preceeding sections of this chapter describe the work involved and the results obtained in the evaluation of the oxygen sag equation parameters. After such an evaluation, it is possible to construct an oxygen sag

TABLE 3-14

ULTIMATE NITRIFICATION DEMANDS, LAG TIMES, AND RATE CONSTANTS

For Passaic River Water in the Vicinity of Pine Brook -
(all results at 20°C)

Sample Type	Date	K_N days	N_o mg/l	t_a days
Composite	7/26/68	0.366	9.9	0.8
Composite	8/19/68	--	>16.8	5.5
Composite	8/22/68	0.205	21.7	2.0
Composite	8/27/68	0.455	13.7	3.4
Composite	8/28/68	0.831	18.5	2.0
Grab, 9:15 a.m.	9/3/68	0.149	19.9	6.8
Grab, 3:15 p.m.	9/3/68	--	> 3.0	2.7
Grab, 9:15 p.m.	9/3/68	9.441	16.1	0.8
Grab, 3:15 a.m.	9/4/68	0.284	15.7	5.0
Grab, 9:15 a.m.	9/4/68	0.231	15.2	3.4
Average*		0.370	16.3	3.0

* Excluding results of composite, 8/19/68, and Grab, 3:15 p.m., 9/3/68

curve based on those results. It was decided to use the summer average flow and temperature, the average values of K_d, L_o, K_n, N_o corrected for temperature differences, the value of C_s at summer average temperature, and the average values obtained for P -R and S. As nitrification represents the least reliable data presented, the oxygen sag curve presented in Figure 3-14 was calculated without a nitrification sink term. These curves represent within the limits of accuracy involved in the calculation of the equation parameter, the average predicted summer dissolved oxygen levels of that stretch of the Passaic River of interest to this project.

ACKNOWLEDGEMENT

The work covered in this chapter reflects work by graduate students Oliver M. Donovan, Thomas T. Tuffey, and Arthur P. Coyan, Jr.

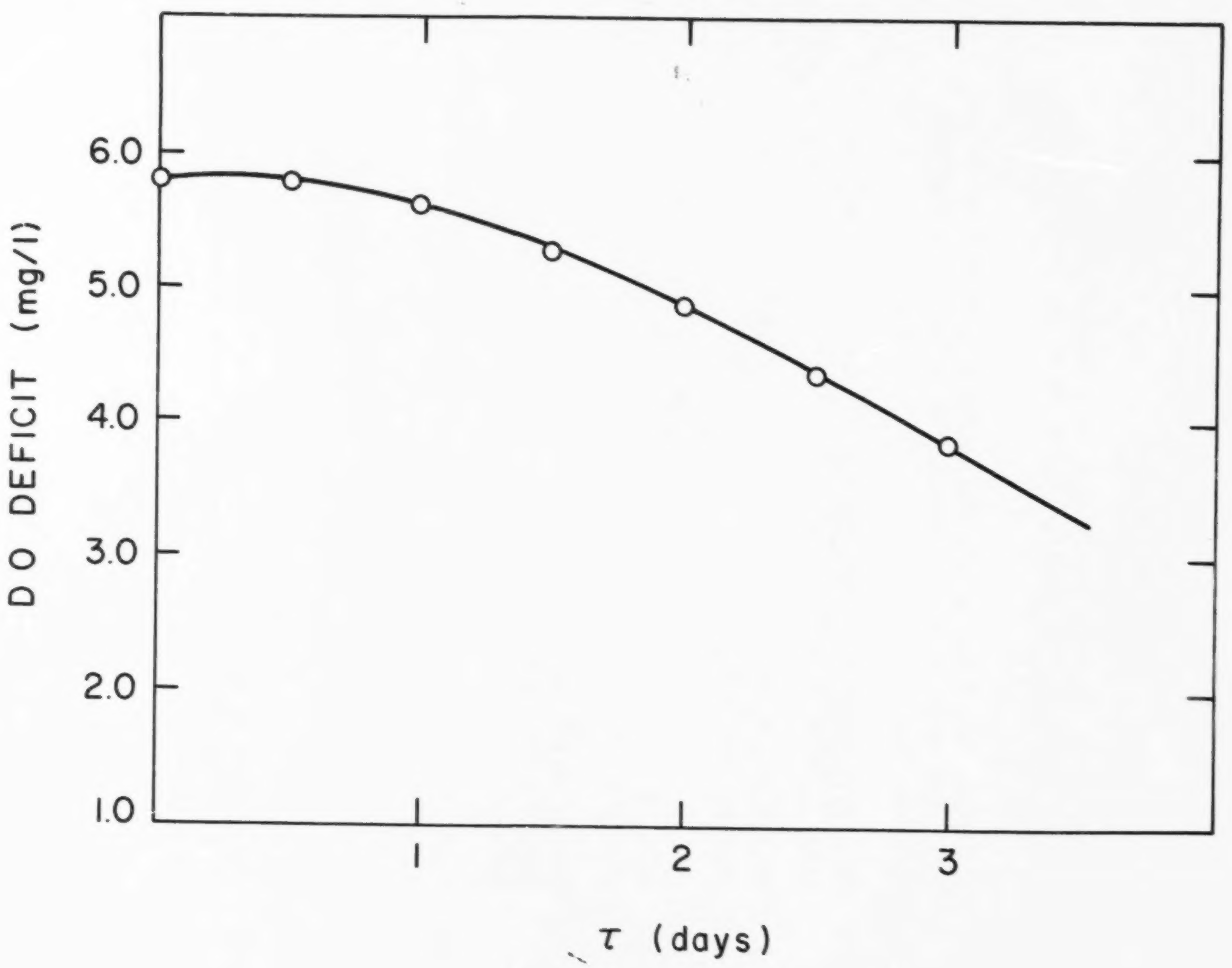

FIGURE 3-14. Calculated O_2 sag profile (Passaic River - 1968)

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Chapter IV

OXYGEN TRANSFER OF AERATORS

Introduction

The oxygen transfer capability of aeration equipment under field conditions is of prime importance to possibilities of induced aeration of polluted rivers. In the past, various techniques for evaluating the performance of aerators in activated sludge and other waste treatment processes have been proposed.(1)(2)(3)(4)(5)(6) However, it was considered that results obtained for aeration tanks or lagoons would not be directly applicable to flowing streams. Therefore, as indicated in Chapter II, field tests were directed towards evaluation of similar results for small rivers.

In this chapter the oxygenation efficiency of surface mechanical and diffuser aerators are compared, and some performance considerations relevant to design are discussed.

Oxygen Transfer Efficiency

The performance of an artificial aeration unit is characterized by the oxygen transfer rate, or the oxygenation efficiency, which is expressed as the amount of oxygen in pounds added by the aerator per shaft horsepower-hour. In a flowing stream the oxygen transfer rate for an aerator operating under steady-state conditions may be expressed by the following equation(7)

$$R_T = \frac{B\ Q\ (C_d - C_u)}{P} \qquad \text{.... Eq. 4-1}$$

where

R_T = oxygen transfer rate at test conditions in lbs. O_2/hp-hr

B = units conversion factor

Q = river discharge in cubic feet per second

C_d = d.o. concentration at downstream (of the aerator) sampling point in mg/l

C_u = d.o. concentration at upstream (of the aerator) sampling point in mg/l

P = power developed by aerator in shaft horsepower. This was calculated from the electric energy actually used during each test.

Laboratory channel studies by Susag et al.(7) showed that Equation 4-1 provides good results as compared to the commonly used unsteady-state equation.* In natural streams, however, the question arises that whether the laboratory channel results are reproductible when the aeration process is subject to various uncontrollable environmental effects. It is, therefore, of interest to apply this equation to the treatment of in-stream data and compare the results with those of laboratory and canal aeration studies. A brief review of these available methods for evaluating aeration performance in streams as well as some theoretical considerations is given in Appendix IV-1.

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*Mathematical relationships similar to equation 4-1 have been described by Kalinske(8). Also Kaplovsky et al.(9) have derived analogous expressions for the analysis of canal aeration data.

The oxygen transfer rate determined by Equation 4-1 is specific for the conditions that obtained during a particular test. For the purpose of comparison this rate must be converted to standard conditions, which are generally defined as follows:

water temperature = 20°C

atmospheric pressure = 760 mm Hg

d.o. concentration = 0 mg/1

water purity = clean

Thus the conversion from test to standard conditions involves corrections for water temperature, atmospheric pressure, water quality and the d.o. level at which the aerator is operating. Procedures for making these corrections are described below.

(i) Correction for Temperature

The oxygen transfer coefficient K_La is a function of water temperature, as can be described by the relationship

$$(K_La)_T = (K_La)_{20} \ (\theta)^{T-20} \qquad \text{.... Eq. 4-2}$$

where

$(K_La)_{20}$ = oxygen transfer coefficient at 20°C

Θ = a temperature dependent coefficient

T = water temperature observed

The temperature coefficient, Θ, has been reported to vary from 1.016 to 1.037.(6)

In this study a Θ value of 1.025 was used, according to O'Connor et al.(10) This value is approximately an average of values cited.

(ii) Correction for Atmospheric Pressure

The effect of atmospheric pressure is primarily on the saturation oxygen concentration, C_s, as described by

$$(C_s)_p = C_s \cdot \frac{p}{760} \qquad \text{.... Eq. 4-3}$$

where $(C_s)_p$ = the saturation concentration of oxygen in water at pressure

p (mm Hg)

C_s = the saturation concentration at one atmospheric pressure

For the diffusion aerator an additional term has been suggested for equation 4-3.(3)

The correction for atmospheric pressure is particularly significant when operating at high elevations.(5) In this study, however, this is not considered because of the fact that the elevation of the aeration site is rather low.

(iii) Correction for Water Quality

The presence of surface-active agents in water has been noted to cause a decrease in the oxygen transfer coefficient.(11) To correct for this a "specific oxygen transfer rate, α, is defined as

$$\alpha = \frac{K_La \text{ in polluted water}}{K_La \text{ in pure tap water}} \qquad \text{.... Eq. 4-4}$$

For different types of imperities, various values of α have been reported. For example, Eckenfelder and Ford(6) observed α values of 0.86 and 1.32 for activated sludge effluents. In the case of river waters, comparatively little data can be found in the literature, except perhaps that Kaplovsky et al.(9) have reported α to range from 0.71 to 0.96 in Chicago canal waters, with an average value of 0.85. Therefore this average value was used in the present analysis.

(iv) Correction for d.o. Level

The oxygen saturation concentration in water is affected by both the quality and the temperature of the water. To make correction for water quality, the following parameter is defined

$$\beta = \frac{\text{solubility of oxygen in polluted water}}{\text{solubility of oxygen in pure water}} \quad \text{.... Eq. 4-5}$$

and therefore

$$C_s' = C_s \times \beta \quad \text{.... Eq. 4-6}$$

where C_s' = saturation oxygen concentration under field conditions Kaplovsky et al.(9) have observed that β varied from about 0.93 to 1.0. Hunter, Ch. III, above, determined that for Passaic River water $\beta = 1.0$, and proposed the use of the following temperature relationship.

$$C_s = 14.652 - 0.41022\,T + 0.007991T^2 - 0.000077774T^3 \quad \text{.... Eq. 4-7}$$

For a temperature T = 20°C, $C_s = 9.02$ mg/1

The correction to standard conditions for d.o. level then can be made by applying the factor $C_s/(C_s - C_m)$ to the transfer rate calculated for test conditions, where C_m is an estimate of the d.o. level at the aerator.

For field aeration tests, it is rather difficult to measure the exact value of oxygen concentration, C_m, at which the aerator is operating. Consequently the driving force $(C_s - C_m)$ cannot be accurately determined, though several methods have been proposed. For example, Susag et al.(7) suggest three ways of estimating C_m based upon three types of assumed d.o. variation between upstream and downstream sampling points. The first type is essentially a step function that has a jump at the aerator, hence $C_m = C_u$; the second is a linear variation and $C_m = 1/2\,(C_u + C_d)$; the third a logarithmic average based on the basic aeration equation. The laboratory results lead them to conclude that the third method might give the best measure of the driving force. In this chapter, all three methods will be used and results compared. For simplicity, the method of estimating C_m with upstream d.o. only will be called method R_S-#1, the one with average upstream and downstream d.o. method R_S-#2 and the one with logarithmic average method R_S-#3.

Based on the foregoing discussions, the conversion of the oxygen transfer rate under test conditions, R_T to that under standard conditions, R_S, is obtained by

$$R_S = R_T \frac{(C_S)_{20}}{[\ (C_S)_T - C_m\ ]\ (TF)\ (\alpha)} \quad \text{.... Eq. 4-8}$$

where

$(C_S)_{20}$ = saturation oxygen concentration at 20°C

$(C_S)_T$ = saturation oxygen concentration at test water temperature T

TF = temperature conversion factor

$= (1.025)^{T-20}$

Computation Procedure and Results

(i) Oxygen Uptake Determinations

The oxygen uptake due to artificial aeration for each test was computed by comparing the d.o. reading downstream and upstream of the aerator. The d.o. readings obtained with longitudinal traverse were first plotted from 3000 ft. upstream, and to 3000 ft. downstream of the mechanical aerator. For the diffuser aerator d.o. readings from 5000 ft. upstream and to 5000 ft. downstream were plotted since these readings were usually taken with a larger interval (e.g. 3000 ft.) than that for the mechanical aerator (e.g. 200 ft.). The oxygen uptake was calculated by the following equation:

$$\text{uptake} = 0.2246\ Q\ (C_d - C_u) \quad \text{.... Eq. 4-9}$$

where

uptake is in lb O_2/hr, and Q, C_d and C_u are defined as earlier.

For the mechanical aerator the oxygen uptake values were determined for three different sampling distances, namely, 200 ft, 1000 ft, and 3000 ft. Since most diffuser aerator uptake values were taken at 1000 ft. or 3000 ft. interval, the efficiency results of the two aerators for the same sampling distance can be compared. In addition it might be of interest to investigate the effect of sampling distance on the results of aerator performance.

A few oxygen uptake values were also determined from lateral d.o. traverse. The mean d.o. for a cross section was computed by using weighted average according to stream velocity readings. Two control sections, one upstream and one downstream would provide the oxygen uptake value.

Figures 4-1 and 4-2 illustrate a typical d.o. profile determined by longitudinal traverse, and a typical cross sectional d.o. readings, respectively. The oxygen uptake values for the mechanical aerator under steady-state conditions together with the field test data are given in Table 4-1, 4-2 and 4-3. For the diffuser aerator the results are listed in Table 4-4.

It can be seen from Table 4-1, 4-2 and 4-3 that for the mechanical aerator the oxygen uptake values obtained with sampling points 200 ft. apart were usually higher than those obtained sampling distance of 1000 ft. and 3000 ft. As was expected the 3000 ft. values were mostly the lowest.

(ii) Comparison of Probe d.o. and Winkler d.o. Values

For each of the d.o. sampling tests a comparison was made of the d.o. readings determined by the dissolved oxygen meter and by the Winklers. It was decided that probe values would be used for determination of relative values over any short period. For absolute values during any test the mean difference between the Winklers and the corresponding probe values during that test would be computed and applied uniformly to all probe values, thus in effect re-calibrating the probe. Where the relationship between Winkler and probe values was not reasonably consistent, the data were not used. Probe readings were

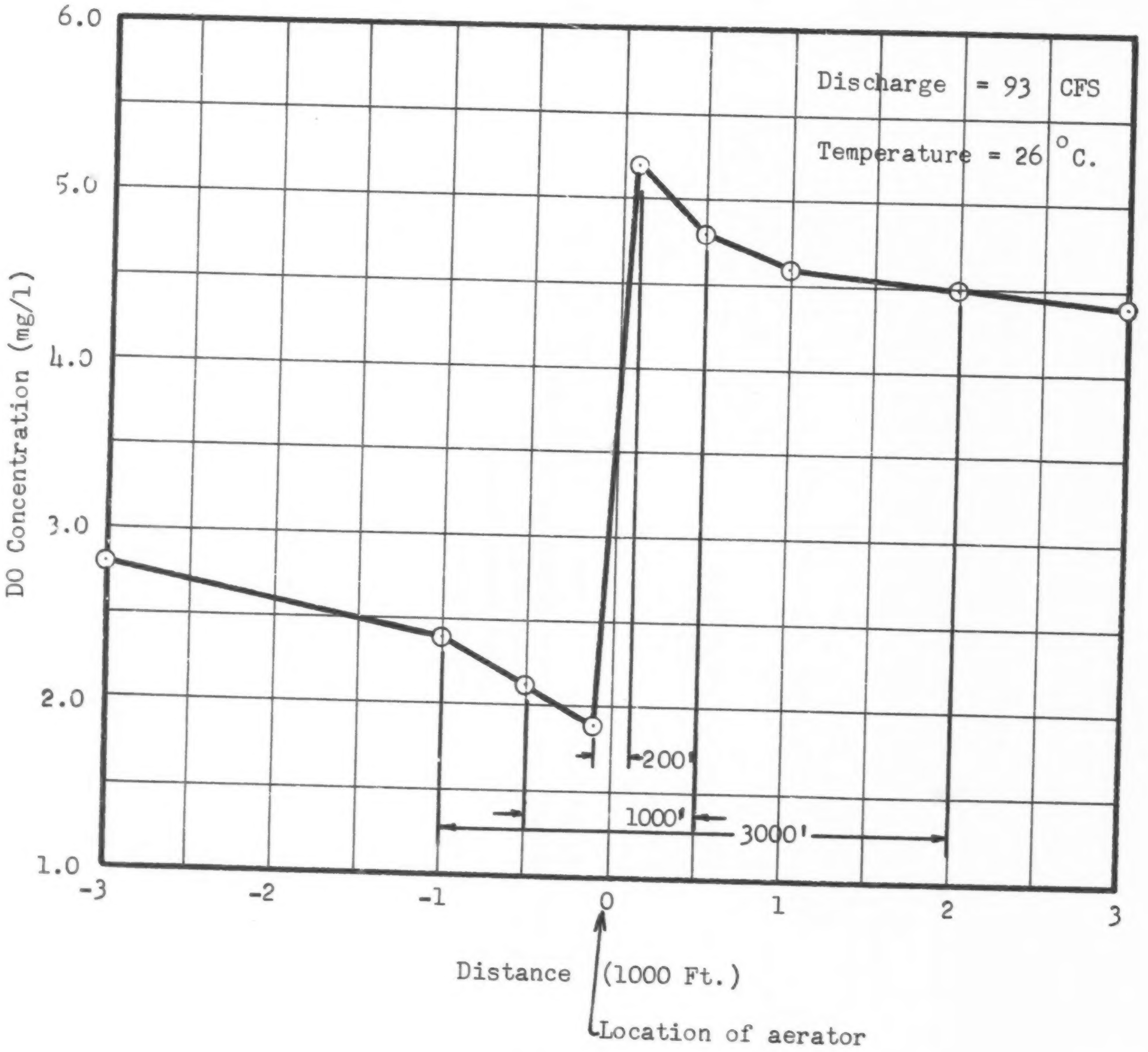


FIG.4-1 A TYPICAL LONGITUDINAL DO TRAVERSE
8/1/68

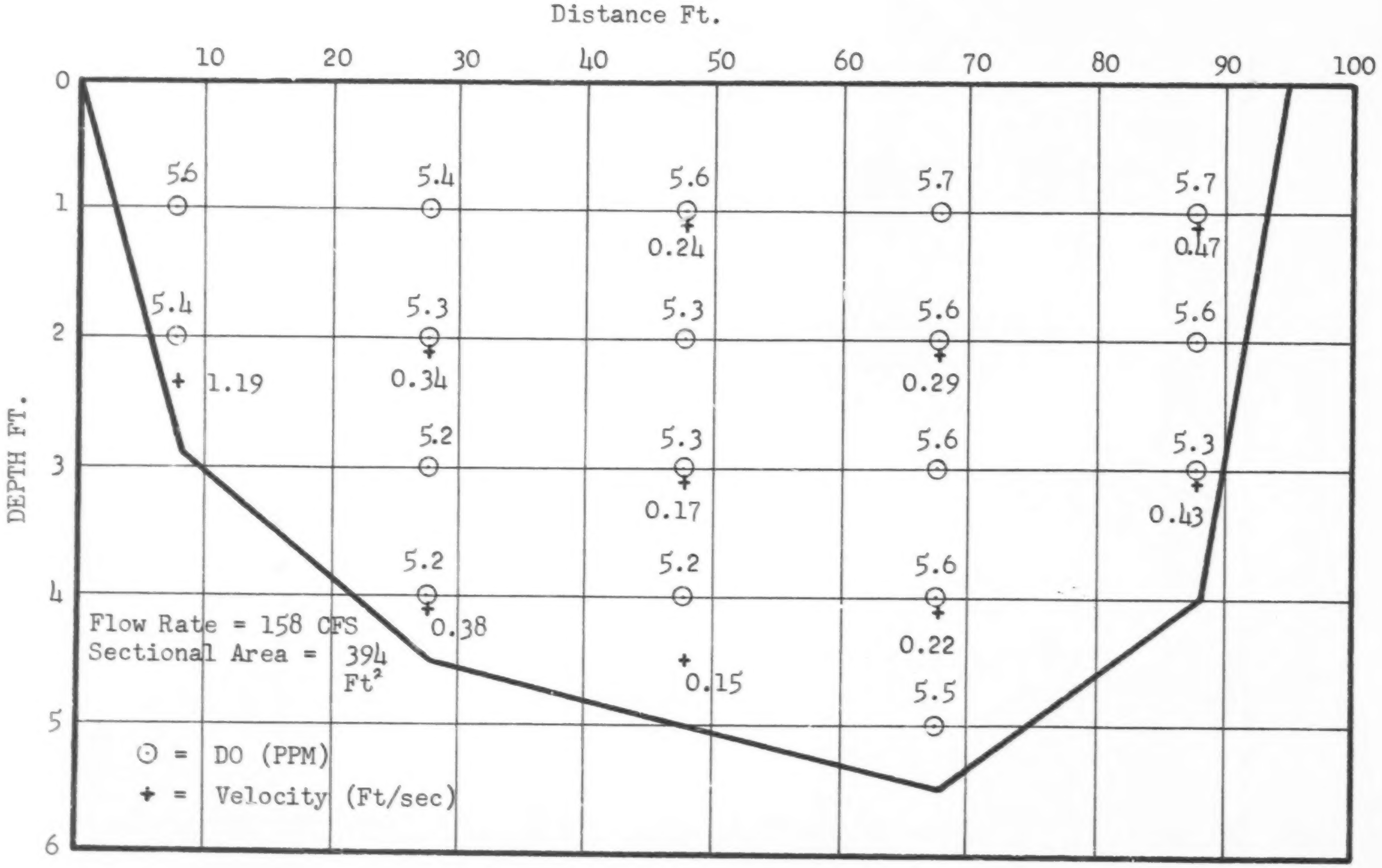


FIG.4-2 DO AND VELOCITY TRAVERSE AT +100 FT.

7/9/68

TABLE 4-1

SUMMARY OF STEADY-STATE FIELD TEST DATA FOR MECHANICAL AERATOR - PART I *

Date	Flow Q (CFS)	Water Temp. (°C)	Shaft Power (HP)	Dissolved Oxygen (ppm)		Oxygen Added (lbs.O_2/hr)
				Upstream	Downstream	
8/9/67	1670	23.0	100.7	0.90	1.50	225.1
8/21	163	23.9	71.1	1.40	3.10	62.2
8/25	122	21.0	65.4	1.60	3.40	49.3
8/29	982	21.5	84.9	2.90	3.50	132.3
8/29	964	22.0	81.3	3.00	3.40	86.6
6/28/68	568	23.0	60.0	2.92	3.65	93.1
7/1	388	24.0	60.2	1.96	2.50	47.1
7/2	324	25.0	77.8	1.29	2.58	93.9
7/10	147	23.5	84.1	4.40	6.34	64.1
7/11	150	22.0	71.6	2.50	4.60	67.4
7/12	133	25.0	76.4	3.60	6.13	71.7
7/16	110	28.8	78.2	3.20	5.60	59.3
7/16	111	25.9	80.8	3.30	5.00	42.4
7/18	128	29.0	77.1	2.80	4.90	60.4
7/18	134	27.7	82.4	2.50	4.50	60.2
7/23	113	29.5	76.6	2.50	5.40	73.6
7/24	110	26.5	75.2	2.10	4.60	61.8
7/24	107	26.5	74.6	2.03	4.60	62.0
7/26	125	25.0	71.6	1.00	4.80	106.7
7/26	133	25.0	72.4	1.80	4.45	79.2
7/29	92	24.8	74.6	2.20	5.60	70.3
7/30	91	23.0	85.7	2.60	6.50	81.8
7/31	93	24.0	85.8	2.20	5.60	71.0
8/1	93	26.0	80.7	1.90	5.20	68.9
8/6	106	25.0	88.3	1.50	4.80	78.6
8/7	110	26.7	81.0	1.60	4.90	81.5
8/7	104	25.0	82.2	1.50	4.90	79.4
8/9	99	26.0	79.3	0.60	5.00	97.8
8/9	98	27.5	78.0	1.50	4.80	72.6
8/22	85	24.9	81.6	1.60	5.30	70.6

---

* DO samples taken at 100 ft. upstream and downstream of the aerator.

TABLE 4-2

SUMMARY OF STEADY-STATE FIELD TEST DATA FOR MECHANICAL AERATOR - PART II *

Date	Flow Q (CFS)	Water Temp. (°C)	Shaft Power (HP)	Dissolved Oxygen (ppm)		Oxygen Added (lbs.O_2/hr.)
				Upstream	Downstream	
8/31/67	602	21.2	92.8	2.32	3.40	146.0
7/9/68	159	24.5	79.5	4.70	5.90	42.9
7/10	147	24.0	84.1	3.70	4.80	36.3
7/11	150	22.0	71.6	2.50	4.50	67.4
7/12	133	25.0	76.4	4.20	6.60	71.7
7/16	111	26.0	80.8	3.15	5.70	63.6
7/16	110	28.7	78.2	3.20	5.40	54.4
7/18	134	27.5	82.4	2.50	4.20	51.2
7/18	128	29.0	77.1	2.80	4.70	54.6
7/23	113	29.5	76.6	2.50	5.40	73.6
7/24	110	26.3	75.2	2.10	4.60	61.8
7/26	125	25.3	71.6	1.10	4.20	87.0
7/29	92	24.8	74.6	2.20	5.60	70.3
7/30	91	23.0	85.7	2.80	6.20	69.5
7/31	93	24.0	85.8	2.10	5.40	68.9
8/1	93	26.0	80.7	2.10	4.80	56.4
8/6	106	25.0	88.3	1.50	4.90	81.0
8/7	110	26.7	81.0	1.20	5.00	93.9
8/7	104	25.0	82.2	1.46	5.10	85.0
8/9	99	26.0	79.3	0.60	4.90	95.5
8/9	98	27.3	78.0	1.60	4.90	72.6
8/22	85	24.9	81.6	1.50	5.00	66.8
8/23	394	25.5	66.2	1.50	2.45	84.1

* DO samples taken 500 ft. upstream and downstream of the aerator.

TABLE 4-3

SUMMARY OF STEADY-STATE FIELD TEST DATA FOR MECHANICAL AERATOR - PART III *

Date	Flow Q (CFS)	Water Temp. (°C)	Shaft Power (HP)	Dissolved Oxygen (ppm) Upstream	Dissolved Oxygen (ppm) Downstream	Oxygen Added (lbs.O_2/hr.)
8/9/67	1620	23.5	90.0	1.12	1.58	167.4
8/31	520	22.0	83.4	2.50	3.35	99.3
7/9/68	159	24.5	79.5	4.60	6.00	50.0
7/10	147	24.0	84.1	3.60	4.45	34.4
7/12	133	25.0	71.7	4.10	6.43	69.6
7/16	111	26.0	80.8	3.00	5.94	73.3
7/16	110	28.6	78.2	3.20	5.35	53.1
7/18	134	27.5	82.4	2.70	4.30	48.2
7/18	128	28.9	77.1	2.90	4.85	56.1
7/23	113	29.4	76.6	2.60	5.00	60.9
7/24	110	26.2	75.2	1.90	4.10	54.4
7/26	125	25.4	71.6	1.20	4.00	78.6
7/29	92	24.8	74.6	2.20	5.10	59.9
7/30	91	23.0	85.7	3.00	6.00	61.3
7/31	93	24.0	85.8	2.10	5.00	60.6
8/1	93	26.0	80.7	2.40	4.50	43.9
8/6	106	25.0	88.3	1.50	4.50	71.4
8/7	110	26.8	81.0	1.70	5.20	86.5
8/7	104	25.0	82.2	1.40	4.70	77.1
8/9	99	26.0	79.3	0.60	4.50	86.7
8/9	98	27.0	78.0	1.60	4.50	63.8
8/22	85	24.9	81.6	1.40	4.50	59.2
8/27	130	23.0	73.1	1.60	3.50	55.5

* DO samples taken 1000 ft. upstream, and 2000 ft. downstream of the aerator.

TABLE 4-4

SUMMARY OF STEADY-STATE FIELD TEST DATA FOR DIFFUSER AERATOR

Date	Flow Q (CFS)	Water Temp. (°C)	Brake Power (HP)	Dissolved Oxygen (ppm)		Oxygen Added (lbs. O_2/hr.)
				Upstream	Downstream	
8/7/68	104	24.0	92.5	3.10	4.70	37.4
8/8	104	27.0	92.5	3.10	4.75	38.5
8/9	98	27.5	92.5	2.90	4.50	35.2
8/9	99	23.5	92.5	2.00	4.60	57.8
8/12	92	24.0	92.5	2.60	5.40	57.9
8/15	85	26.5	75.5	2.10	4.50	45.8
8/16	83	25.0	92.5	1.40	4.46	57.0
8/19	106	24.5	75.5	0.25	1.75	35.7
8/20	97	26.5	60.0	0.45	2.90	53.2
8/21	93	26.0	75.5	3.90	4.70	16.7
8/23	366	25.0	75.5	1.10	1.85	61.7

generally close to those of the Winklers due to frequent calibration. The mean deviation of each individual probe reading about the mean difference for a given test ranged mostly from 0 to 0.20 ppm. The average of the mean deviations for all tests was about 0.14 ppm, which shows that the probe readings were fairly consistent.

(iii) Analysis of Accuracy of Center Line 1-ft d.o. Readings

As mentioned earlier in Chapter II, for the longitudinal d.o. traverse the measurements were made along the center line of the river and at a depth of one foot. It is therefore necessary to compare this 1-ft. center line d.o. reading with the more accurate section mean d.o. determined by lateral traverses, in order to check the applicability of the former as an index of the section mean d.o.

For all the lateral traverses the difference between section mean d.o. and the corresponding 1-ft. center-line d.o. values was computed. The difference was found to vary from -0.14 to +0.44 ppm, with an average difference of +0.05 ppm. The mean deviation of each individual difference about the mean was 0.12 ppm. The above results include cases when the aerator was in operation.

An example is given in Figure 4-2 (aerator on). The weighted mean d.o. for the section is 5.5 ppm, whereas the 1-ft. center-line d.o. readings is 5.6 ppm. Also it can be seen that the d.o. values across the section are fairly uniform, indicating that the mixing action at a distance of 100 ft. downstream of the aerator is rather complete.

The foregoing results show that the 1-ft. center-line d.o. reading could be used as a good estimate of the mean d.o. for the section. Even though the Passaic River is a very slow stream, with marked photosynthesis in upper layers and benthal demand from the bottom, it seems that natural turbulence is sufficient to prevent stratification of flow.

(iv) Oxygen Transfer Rates

The oxygen transfer rates under test conditions were computed by equation 4-1, and are summarized in

Table 4-7 for the mechanical aerator, and in Table 4-8 for the diffuser aerator. Equation 4-8 was then employed to convert these transfer rates to standard conditions. These results were also shown in Tables 4-7 and 4-8. In order to facilitate the computation, the following relation is defined

$$R_S = R_T \times F \qquad \text{.... Eq. 4-10}$$

where

F = conversion factor from test to standard conditions.

Since F is a function of the following:

$$F = F(C_m, C_s, T, \alpha, \beta) \qquad \text{.... Eq. 4-11}$$

for given C_s, α and β values, F can be expressed as a function of C_m and T. In this study, F is written as

$$F = \frac{9.02}{(C_s - C_m)\,(1.025)^{T-20}\,(0.85)} \qquad \text{.... Eq. 4-12}$$

Thus values of F can be computed and tabulated (Table 4-5), or shown graphically (Figure 4-3). For observed temperature and estimated C_m values, F can be found easily either from Table 4-5 or Figure 4-3, thus simplifies the procedure.

With charts like Figure 4-3, made for particular river systems, the design process can be greatly facilitated.

It should be noted that the results given in Table 4-7 for the mechanical aerator were based on samples taken 200 ft. apart, and in Table 4-8 for the diffuser aerator 3000 ft. apart. In order to make the proper comparison of the two aerators it was necessary to reduce the longer distance results to equivalent values at 200 ft.

Table 4-6 gives the comparison of oxygen transfer rates under steady-state conditions, R_s, for different sampling distances for the mechanical aerator. In this analysis method R_S-#1 and R_S-#2 were applied. The results indicate that, for R_S-#2, on the average R_S for

TABLE 4-5

CONVERSION FACTOR F

VS

WATER TEMPERATURE, T_w, AND DO CONCENTRATION C_m

C_m ppm	T_w °C 0	5	10	15	20	22½	25	30	35	40
0.0	1.19	1.20	1.21	1.20	1.18	1.16	1.15	1.11	1.09	1.07
0.5	1.23	1.25	1.26	1.26	1.24	1.23	1.22	1.19	1.17	1.17
1.0	1.27	1.30	1.32	1.33	1.32	1.32	1.31	1.29	1.27	1.28
1.5	1.32	1.36	1.39	1.41	1.41	1.41	1.40	1.40	1.40	1.42
2.0	1.37	1.42	1.47	1.49	1.51	1.52	1.52	1.52	1.54	1.60
2.5	1.43	1.49	1.55	1.59	1.63	1.64	1.65	1.68	1.72	1.82
3.0	1.49	1.57	1.64	1.71	1.76	1.79	1.81	1.87	1.95	2.12
3.5	1.56	1.65	1.75	1.84	1.92	1.96	2.00	2.10	2.25	2.54
4.0	1.63	1.75	1.87	1.99	2.11	2.18	2.24	2.41	2.66	3.16
4.5	1.71	1.85	2.01	2.17	2.35	2.44	2.55	2.82	3.26	4.17
5.0	1.80	1.97	2.17	2.38	2.64	2.78	2.95	3.40	4.19	6.16

Note:

$$F = \frac{RS}{RT} = \frac{9.02}{(C_s - C_m)(TF)\alpha}$$

where

RS = oxygen transfer rate under standard conditions
RT = oxygen transfer rate under test (field) conditions
C_s = saturation oxygen value at water temperature
C_m = estimated oxygen level at which the aerator is in operation
α = 0.85
TF = $(1.025)^{T_w - 20}$

TABLE 4-6

COMPARISON OF OXYGEN TRANSFER RATES

DETERMINED UNDER STEADY-STATE FOR DIFFERENT SAMPLING POSITIONS

MECHANICAL AERATOR

Date	Flow (CFS)	Oxygen Transfer Rates Under Standard Conditions, lbs.O_2/hp-hr					
		200 ft.		1000 ft.		3000 ft.	
		RS_1*	RS_2**	RS_1*	RS_2**	RS_1*	RS_2**
7/10/68	147	1.84	2.43	0.90	1.02	0.84	0.94
7/12/	133	1.93	2.72	1.79	2.56	1.69	2.37
7/16	111	1.02	1.24	1.48	2.00	1.65	2.34
7/16	110	1.94	2.02	1.34	1.79	1.31	1.73
7/18	128	1.39	1.78	1.26	1.57	1.32	1.66
7/18	134	1.22	1.50	1.03	1.23	1.01	1.20
7/23	113	1.61	2.26	1.61	2.26	1.36	1.80
7/24	110	1.27	1.61	1.27	1.61	1.08	1.32
7/26	125	1.95	2.65	1.61	2.06	1.47	1.85
7/29	92	1.48	2.06	1.48	2.06	1.26	1.66
7/30	91	1.59	2.41	1.40	2.00	1.28	1.76
7/31	93	1.30	1.79	1.24	1.68	1.09	1.42
8/1	93	1.28	1.75	1.08	1.40	0.88	1.09
8/6	106	1.25	1.66	1.29	1.73	1.14	1.47
8/7	110	1.43	1.94	1.55	2.16	1.54	2.15
8/7	104	1.36	1.82	1.44	1.98	1.30	1.72
8/9	99	1.52	2.16	1.16	1.50	1.35	2.10
8/9	98	1.34	1.83	1.32	1.80	1.16	1.51
8/22	85	1.23	1.72	1.15	1.56	1.00	1.30
Average		1.47	1.97	1.34	1.79	1.25	1.64

* Based on upstream DO.

** Based on average of upstream and downstream DO.

TABLE 4-7

SUMMARY OF OXYGEN TRANSFER RATES UNDER TEST CONDITIONS (RT), AND UNDER STANDARD CONDITIONS (RS) FOR MECHANICAL AERATOR UNDER STEADY-STATE OPERATION

DATE	FLOW (CFS)	RT ($lbs.O_2/hp-hr$)	RS ($lbs.O_2/hp-hr$) *		
			RS_1	RS_2	RS_3
8/9/67	1670	2.23	2.90	3.02	3.02
8/9	1620	1.86	2.98	3.07	3.15
8/21	163	0.87	1.21	1.39	1.40
8/25	122	0.75	1.08	1.23	1.24
8/29	982	1.56	2.72	2.87	2.87
8/29	964	1.07	1.90	1.97	1.98
8/31	602	1.57	2.75	3.00	3.00
6/28/68	568	1.55	2.74	2.93	2.93
7/1	388	0.78	1.18	1.23	1.22
7/2	324	1.21	1.64	1.81	1.82
7/9	159	0.54	1.59	1.90	1.93
7/9	159	0.63	1.99	2.49	2.53
7/10	147	0.76	1.84	2.43	2.52
7/11	150	0.94	1.54	1.86	1.89
7/12	133	0.94	1.93	2.72	2.80
7/16	110	0.76	1.94	2.02	2.12
7/16	111	0.91	2.00	2.84	3.05
7/18	128	0.78	1.39	1.78	1.83
7/23	113	0.96	1.61	2.26	2.40
7/24	110	0.82	1.27	1.61	1.66
7/24	107	0.84	1.28	1.63	1.66
7/26	125	1.49	1.95	2.65	2.77
7/26	133	1.09	1.61	2.03	2.08
7/29	92	0.94	1.48	2.06	2.18
7/30	91	0.95	1.59	2.41	2.66
7/31	93	0.83	1.30	1.79	1.89
8/1	93	0.85	1.28	1.75	1.83
8/6	106	0.92	1.42	1.90	1.98
8/7	110	1.01	1.43	1.94	2.03
8/7	104	0.97	1.36	1.82	1.90
8/7	110	1.16	1.71	2.38	2.52
8/9	99	1.23	1.52	2.16	2.30
8/9	98	0.96	1.34	1.83	1.90
8/22	85	0.87	1.23	1.72	1.81
8/27	130	0.76	1.32	1.55	1.56
	Average	1.04	1.71	2.12	2.18

* RS_1 was based on upstream DO,
RS_2 the average of upstream and downstream DO, and
RS_3 the aeration equation.

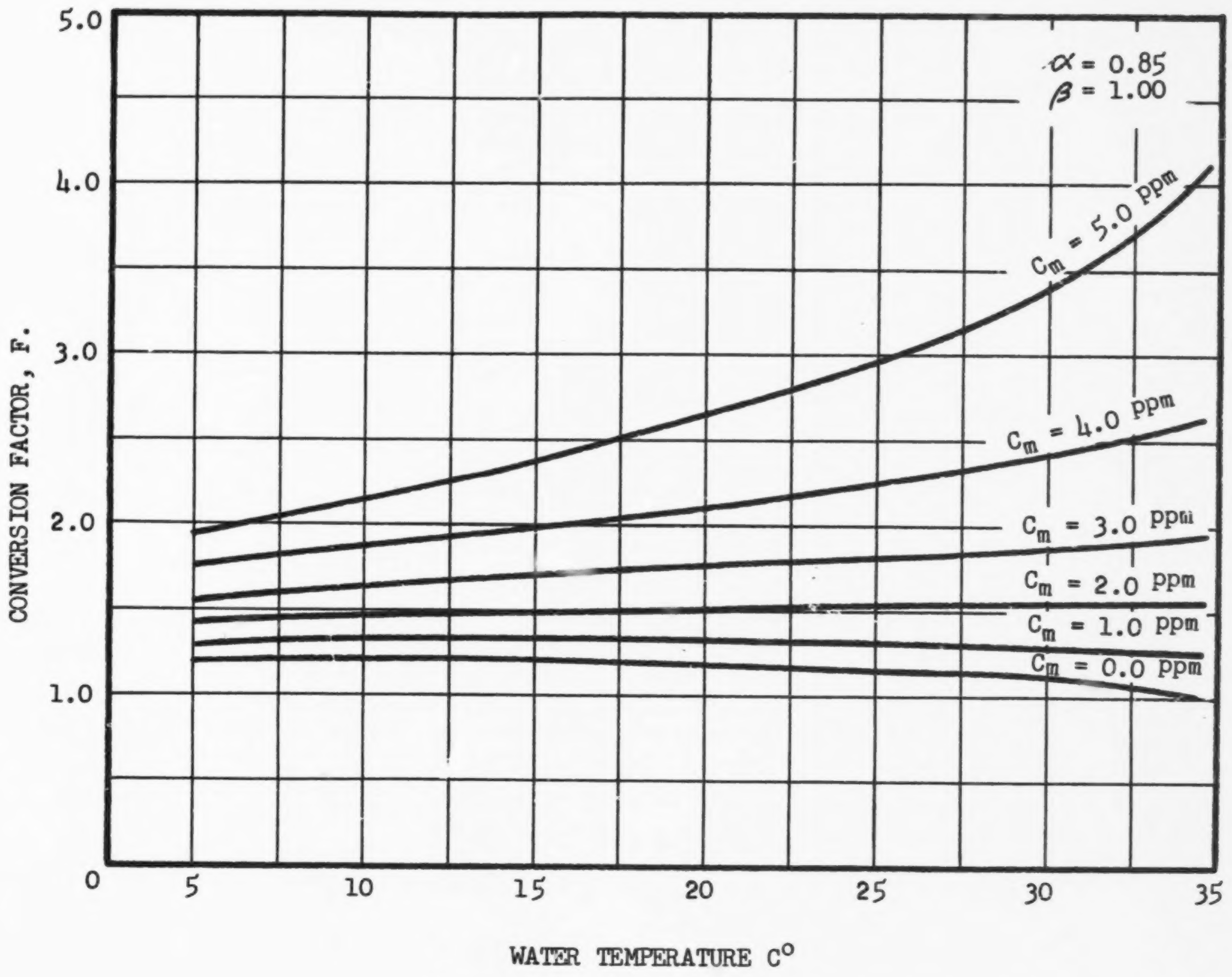
TABLE 4-8

SUMMARY OF OXYGEN TRANSFER RATES UNDER TEST CONDITIONS (RT), AND UNDER STANDARD CONDITIONS (RS) FOR DIFFUSER AERATOR UNDER STEADY-STATE OPERATION

(Sampling Distance 3000 Ft.)

DATE	FLOW (CFS)	RT (lbs.O_2/hp-hr)	RS (lbs.O_2/hp-hr) *		
			RS_1	RS_2	RS_3
8/7/68	104	0.40	0.74	0.88	0.89
8/8	104	0.42	0.78	0.94	0.96
8/9	98	0.38	0.69	0.82	0.83
8/9	99	0.62	0.95	1.19	1.22
8/12	92	0.63	1.05	1.39	1.44
8/15	85	0.61	0.94	1.18	1.21
8/16	83	0.62	0.85	1.10	1.13
8/19	106	0.47	0.56	0.62	0.62
8/20	97	0.89	1.08	1.28	1.35
8/21	93	0.22	0.49	0.54	0.55
8/23	366	0.82	1.08	1.14	1.15
	Average	0.55	0.84	1.01	1.03

* RS_1 was based on upstream DO,
RS_2 the average of upstream and downstream DO,and
RS_3 the aeration equation.



CONVERSION FACTOR, F, vs. WATER TEMPERATURE, T_w, and

D.O. LEVEL AT THE AERATOR, C_m

FIG.4-3

sampling distance 200 ft. is 10% higher than that of 1000 ft., and 20% higher than that of 3000 ft. The results are also shown graphically in Figures 4-4 and 4-5. These differences are probably accounted for by the accelerated sag curve downstream resulting from the induced aeration process. Undoubtedly, more accurate results will be obtained by sampling as close as practicable above and below the aerator.

In this study the 200 ft. sampling distance (100 ft upstream and 100 ft. downstream of the aerator) was considered to be the most representative. As shown in Figures 4-6 and 4-7, which show a circular d.o. traverse and a longitudinal d.o. traverse, respectively, to as close to the aerator as about 30 ft., the "aeration zone" seems to start at approximately 60 ft. upstream of the aerator. The downstream d.o. values are almost uniform to 100 ft. from the aerator. Therefore, the 200 ft sampling distance could be considered "as close together as is commensurate with eliminating sampling of aerated water upstream and with allowing sufficient mixing before downstream sampling."(9)

Another important problem in the study of aerator performance is the accurate estimate of the d.o. level at which oxygen transfer is occuring, which was designated earlier as C_m. It is rather difficult, if not impossible, to observe the exact "driving force" for such a system, since actually oxygen transfer to the water occurs both in the sprayed liquid and the turbulent mixing zone.(6) In this study the three methods described by Susag et al.(7) were used, as mentioned earlier. From Table 4-7 for the mechanical aerator the oxygen transfer rate by method R_S-#2 averages about 24% higher than that by method R_S-#1, but only about 3% lower than that by method R_S-#3. From Table 4-8 for the diffuser aerator, method R_S-#2 gives on the average about 20% higher transfer rate than method R_S-#1, while only about 2% lower than method R_S-#3. Consequently it was concluded that the use of the arithmetic average of the 100 ft upstream and 100 ft downstream d.o. readings (method R_S-#2) could provide satisfactory oxygen transfer results.

Table 4-7 and 4-8 show that the average oxygen transfer rates under standard conditions over the observed

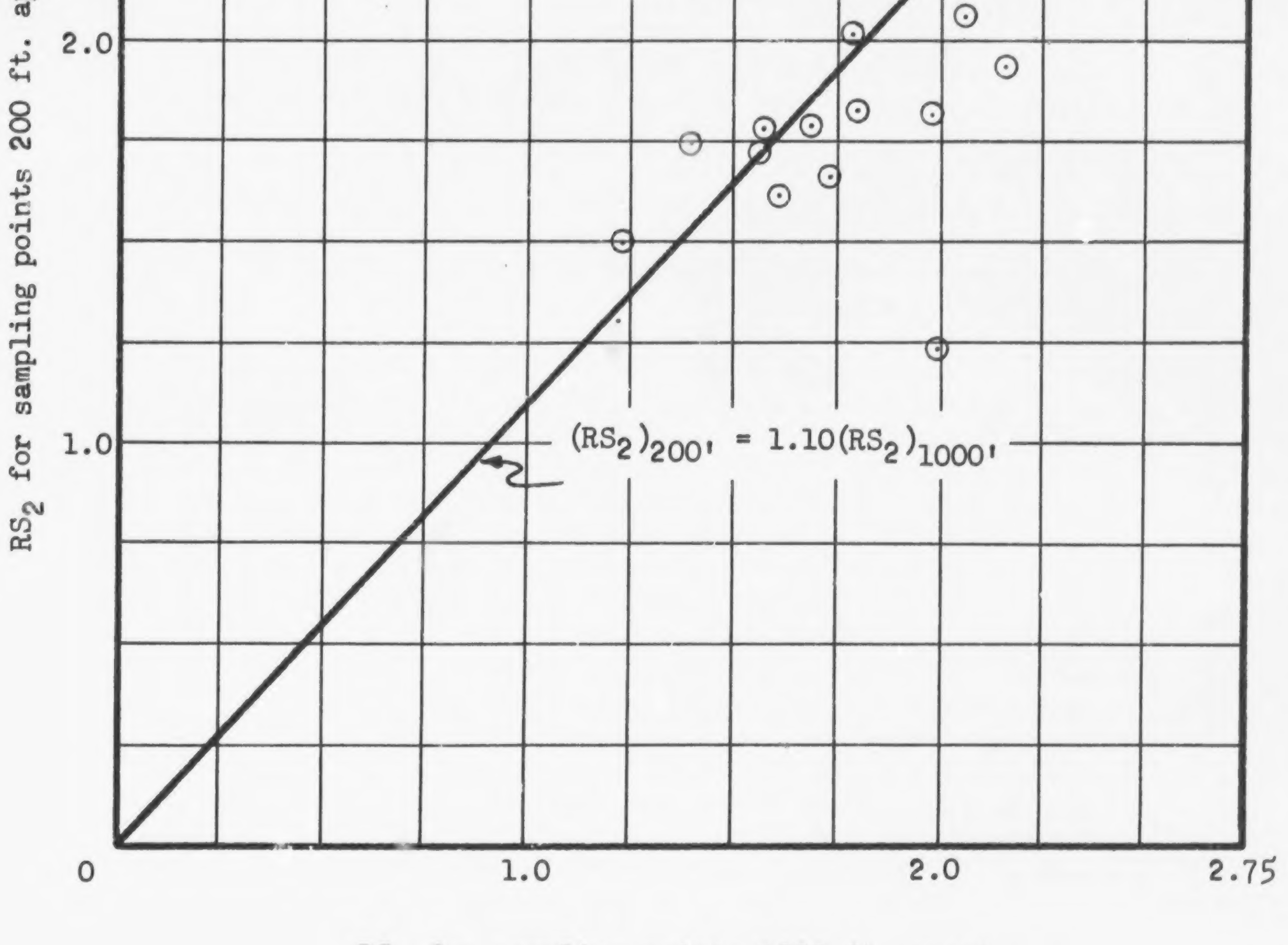


FIG. 4-4

COMPARISON OF OXYGEN TRANSFER RATES UNDER STANDARD CONDITIONS,

RS, FOR DIFFERENT SAMPLING POSITIONS

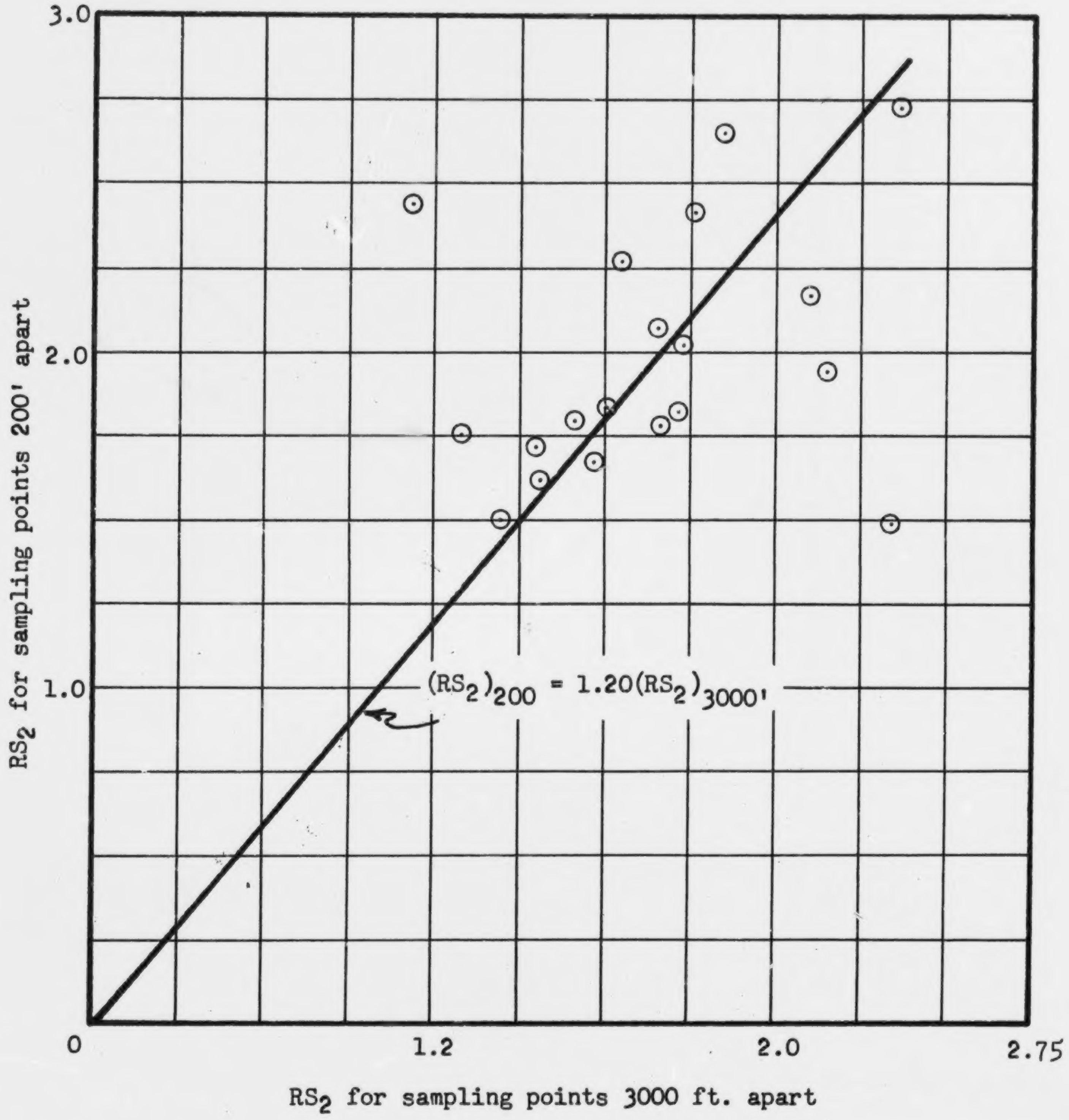
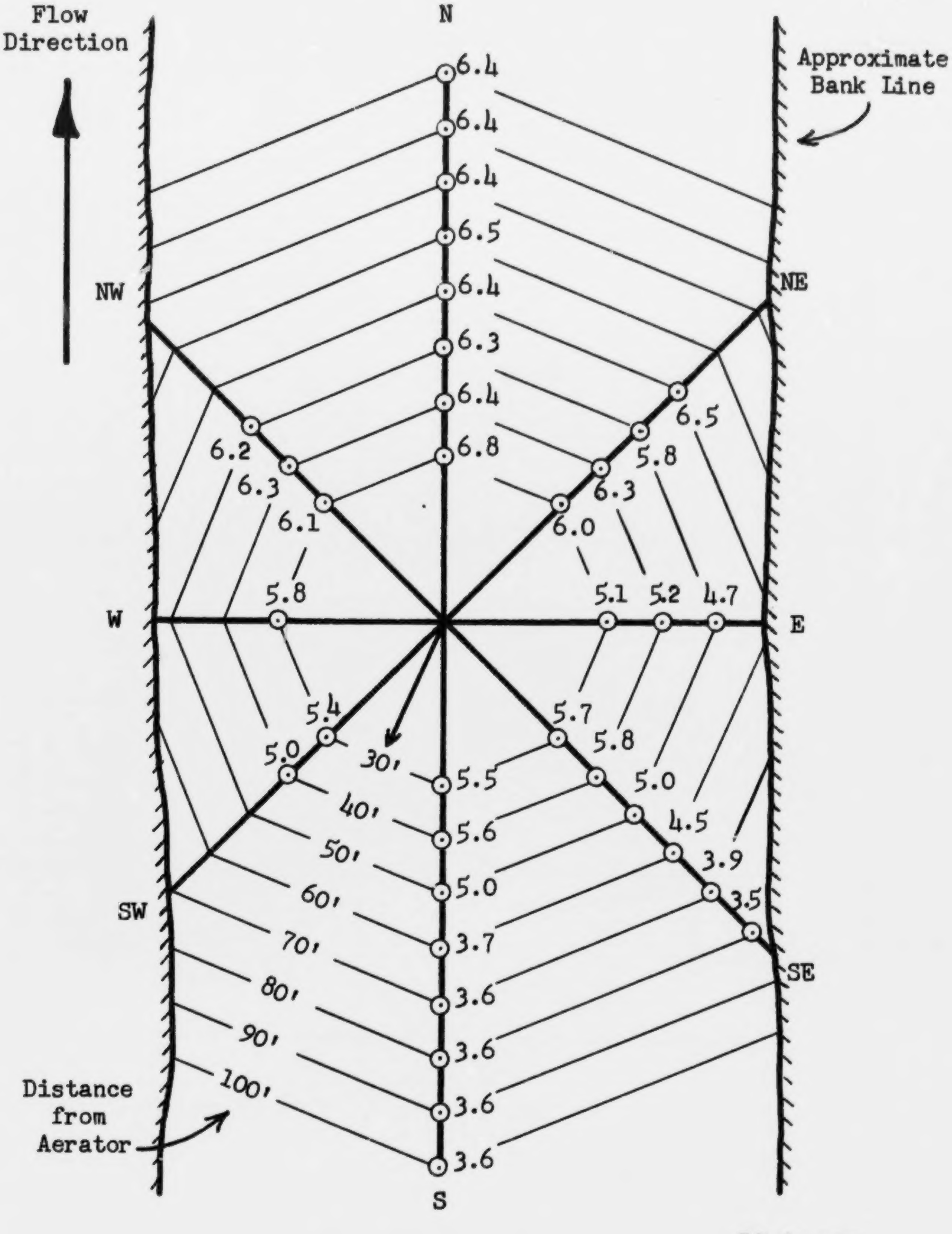


FIG. 4-5

COMPARISON OF OXYGEN TRANSFER RATES UNDER STANDARD CONDITIONS, RS, FOR DIFFERENT SAMPLING POSITIONS


Flow Direction
N
Approximate Bank Line
NW
NE
6.4
6.4
6.4
6.5
6.4
6.3
6.4
6.8
6.2
6.3
6.1
6.5
5.8
6.3
6.0
W
5.8
5.1
5.2
4.7
E
5.4
5.0
5.7
5.8
5.0
4.5
3.9
3.5
30'
40'
50'
60'
70'
80'
90'
100'
5.5
5.6
5.0
3.7
3.6
3.6
3.6
3.6
SW
SE
Distance from Aerator
S

DO in ppm
Flow Rate = 93 CFS

FIG. 4-6

CIRCULAR DO TRAVERSE AROUND AERATOR UNDER STEADY STATE

July 31, 1968

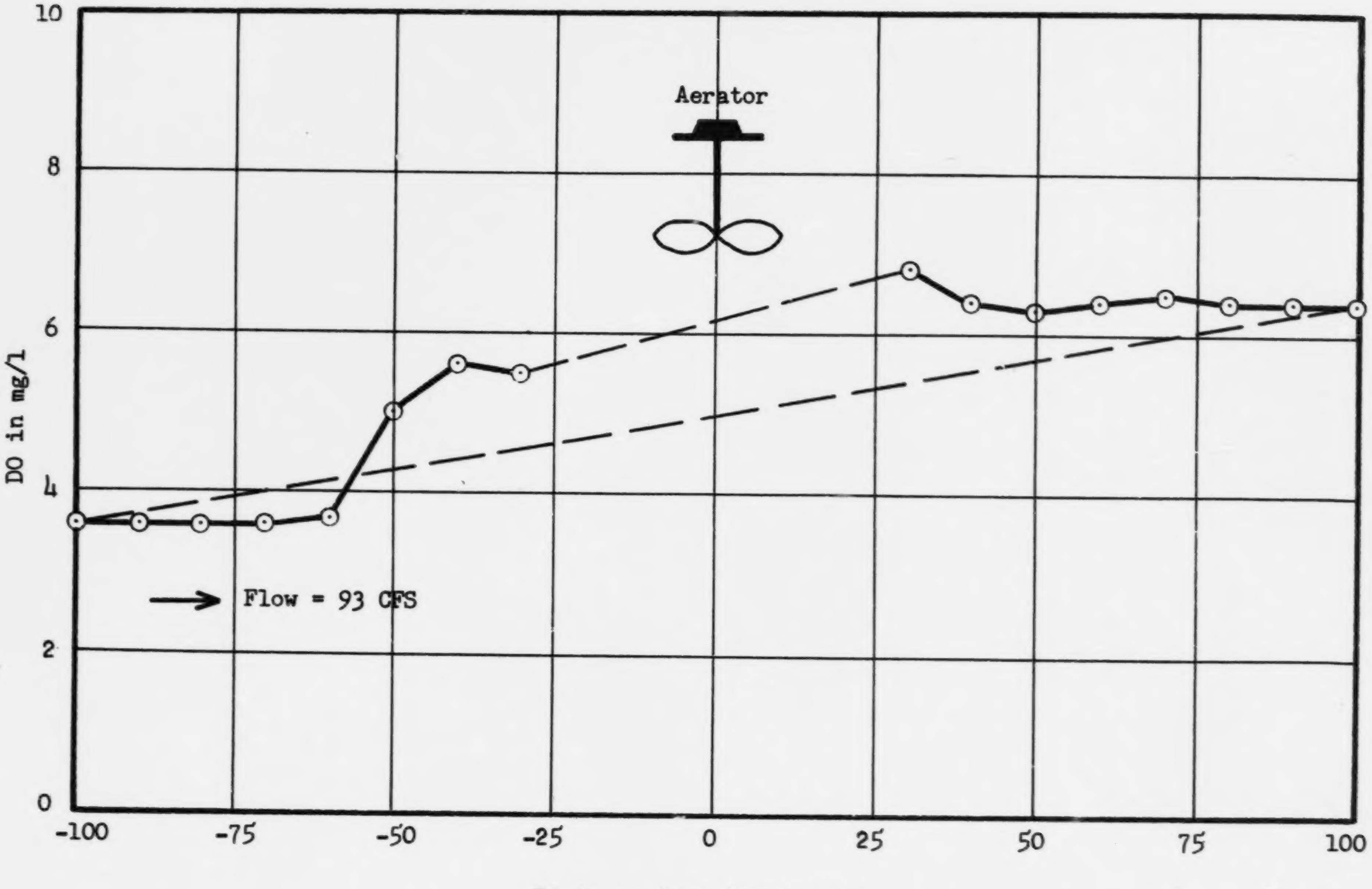


FIG. 4-7

LONGITUDINAL DO TRAVERSE IN THE VICINITY OF THE MECHANICAL AERATOR flow range (85 to 1670 cfs) are approximately 2.1 lb O_2/hp-hr for the mechanical aerator, and 1.01 lb O_2/hp-hr for the diffuser aerator, both based on method R_S-#2. If the relationship between oxygen transfer rates determined by different sampling distances for the mechanical aerator could be applied to the diffuser aerator, the 1.01 lb O_2/hp-hr rate at 3000 ft should be increased to 1.2 lb O_2/hp-hr at 200 ft, to be comparable with that of the mechanical aerator.

The above results indicate that the mechanical aerator gives generally significantly lower oxygen transfer rates operating in streams than operating in aeration tanks or lagoons. According to Eckenfelder and Ford(6), most surface aerators transfer 3.2 - 3.8 lb O_2/hp-hr in activated sludge plants and aerated lagoons. Thus in the river the efficiency drops to about 60% as much. This difference may be due to the greater effect of recirculation of aerated water, as indicated in Chapter I. Also, the inaccuracy of the estimating of the "driving force," the variation in the water quality, and other factors such as power fluctuations (discussed in Chapter II) may contribute to the decrease in aeration efficiency.

For unsteady-state conditions, several computations were also made using equation 4-19 as described in Appendix IV-1. Although quite approximate, the results were found to be comparable with those of steady-state conditions. An example of the computation is given in Table 4-9. The actual measurements are also shown in Figure 4-8.

Such measurements as that shown in Figure 4-4 can also provide steady-state oxygen transfer rate estimates, which are considered to be more reliable than the unsteady-state computation given above. Under the influence of the aerator, the oxygen content of the river increases from the base level to a constant level and then drops to another base level when the aerator is turned off. The average of these two changes in d.o. content, Δ_1 and Δ_2, gives an estimate of the oxygen uptake, and hence the oxygen transfer rate. These data show clearly in many cases the effects of changes in d.o. level from upstream during the period of the test.

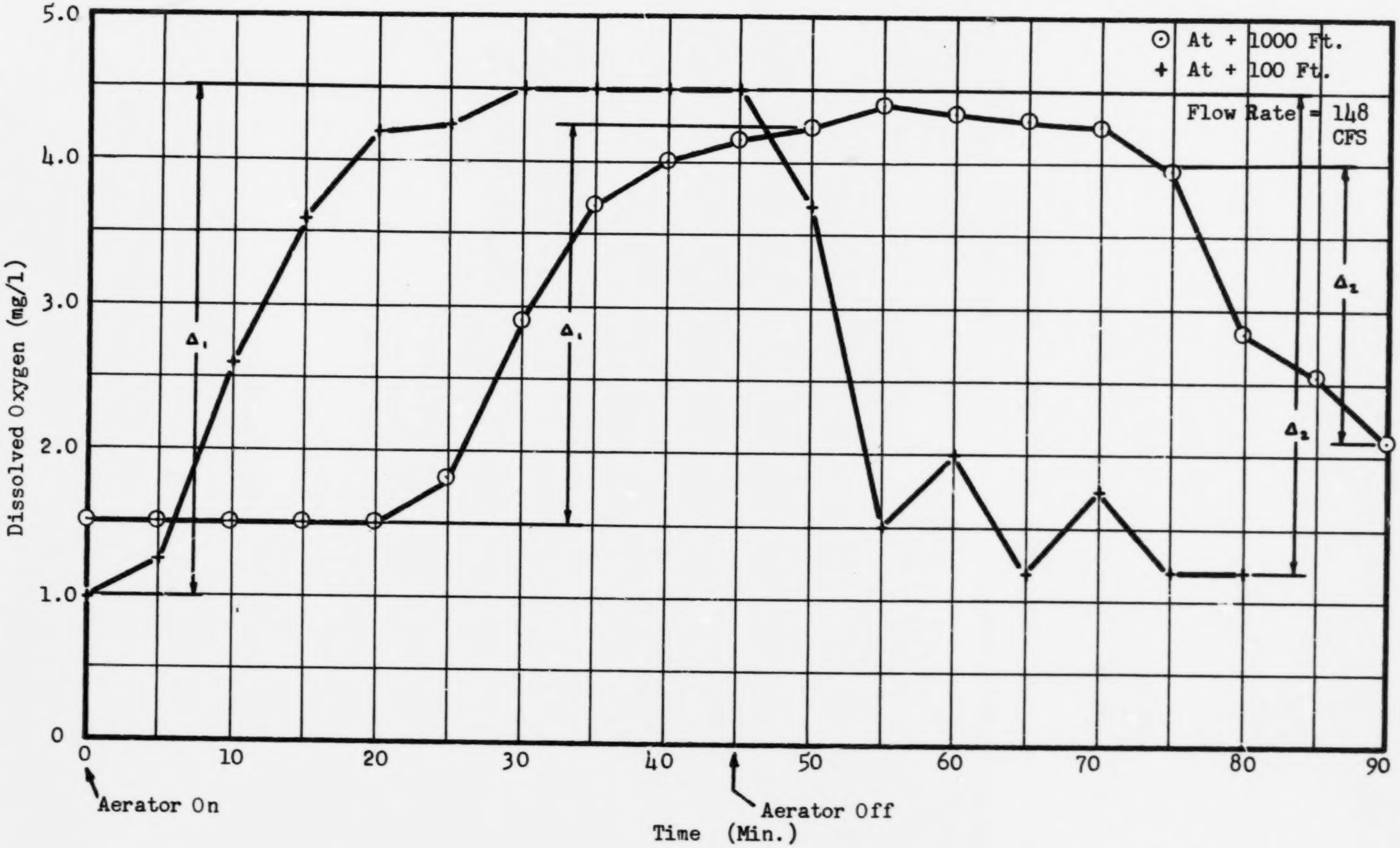


FIG. 4-8 Unsteady State DO Traverse

7/26/68

TABLE 4-9

AN EXAMPLE OF THE UNSTEADY-STATE FIELD TEST DATA

Mechanical Aerator*

Time (Min.)	DO (ppm)	DO Deficit (ppm)	Time (Min.)	DO (ppm)	DO Deficit (ppm)
0	1.00	7.31	16	3.90	4.41
2	1.00	7.31	18	4.00	4.31
4	1.10	7.21	20	4.20	4.11
6	1.40	6.91	22	4.40	3.91
8	1.75	6.56	24	4.25	4.06
10	2.60	5.71	26	4.30	4.01
12	3.20	5.11	28	4.35	3.96
14	3.75	4.56	30	4.50	3.81

* Date: July 26, 1968 Location: 100 ft. downstream of aerator
Flow: 148 c.f.s Water Temperature: 24° C
Estimated volume between sampling points: 110,000 cu. ft.
Saturation oxygen level: 8.31 ppm.

To investigate the effect of the river flow rate on the performance of the aerator, the oxygen transfer rates under standard conditions were plotted against the corresponding flow rate. The results are shown in Figures 4-9 4-10 and 4-11. It can be seen that there is a general increase of efficiency with flow rate in all cases, though not very clearly indicated for lack of data at high flows. Also the figures show that at a constant flow rate there seems to be a wide range of variation in oxygen transfer rates. For example, in Figure 4-9 at a flow rate of 120 cfs the oxygen transfer rate varies from about 1.2 lb O_2/hp-hr to 2.8 lb O_2/hp-hr for standard conditions. While the cause of this variation clearly requires further study, several possible factors might be listed below:

(a) change in water quality, i.e. there is variation in the α -factor.

(b) dispersion effect due to non-uniform velocity distribution across the river section, as can be seen in Figure 4-2.

(c) irregularity in power consumption fluctuations.

(d) sampling errors.

Figure 4-11 illustrates the lower efficiency of the diffuser aerator as compared to the mechanical aerator. The former transfers only about 60% of the oxygen transferred by the latter, when reduced to the same sampling distance of 200 ft.

Considerations Relevant to Systems Design

As previously mentioned, it was found that the oxygen transferrate for the mechanical aerator has an increasing trend when river flow increases from low to moderately high values. The average oxygen transfer rate under standard conditions over the range of flow encountered was approximately 2.1 lb O_2/hp-hr, based on the assumption that $C_m = 1/2\ (Cu + Cd)$. If it is assumed that $C_m = C_u$, the average value becomes 1.7 lb O_2/hp-hr.

Suppose that a small river with d.o. level 1.0 mg/l is to be aerated by a mechanical aerator so that the d.o. level is raised to 5.0 mg/l. the normal flow in

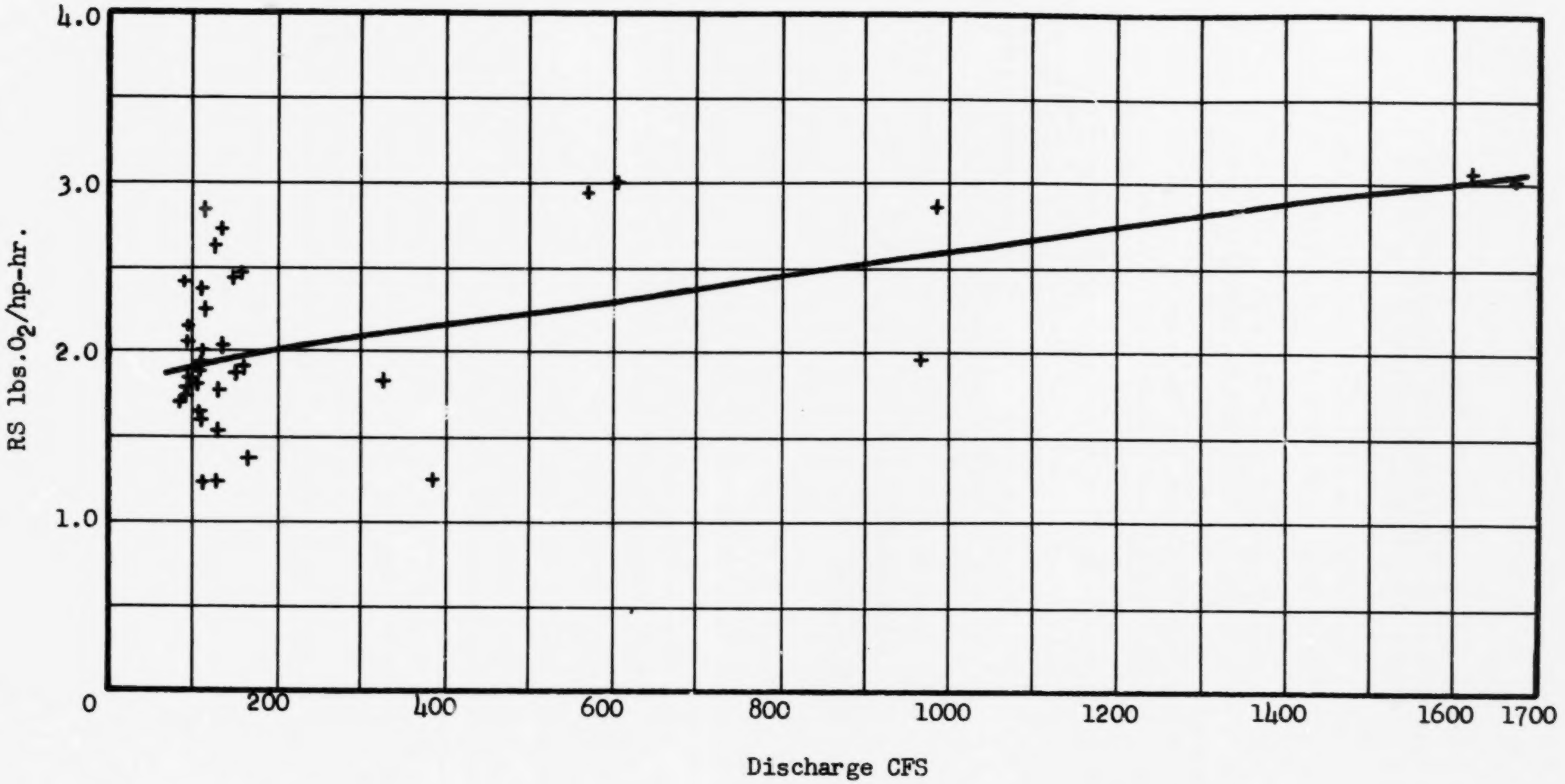


FIG. 4-9

OXYGEN TRANSFER RATE VS FLOW MECHANICAL AERATOR

$(C_m = \frac{1}{2}(C_u + C_d))$ SAMPLING POINT 200 FT. APART

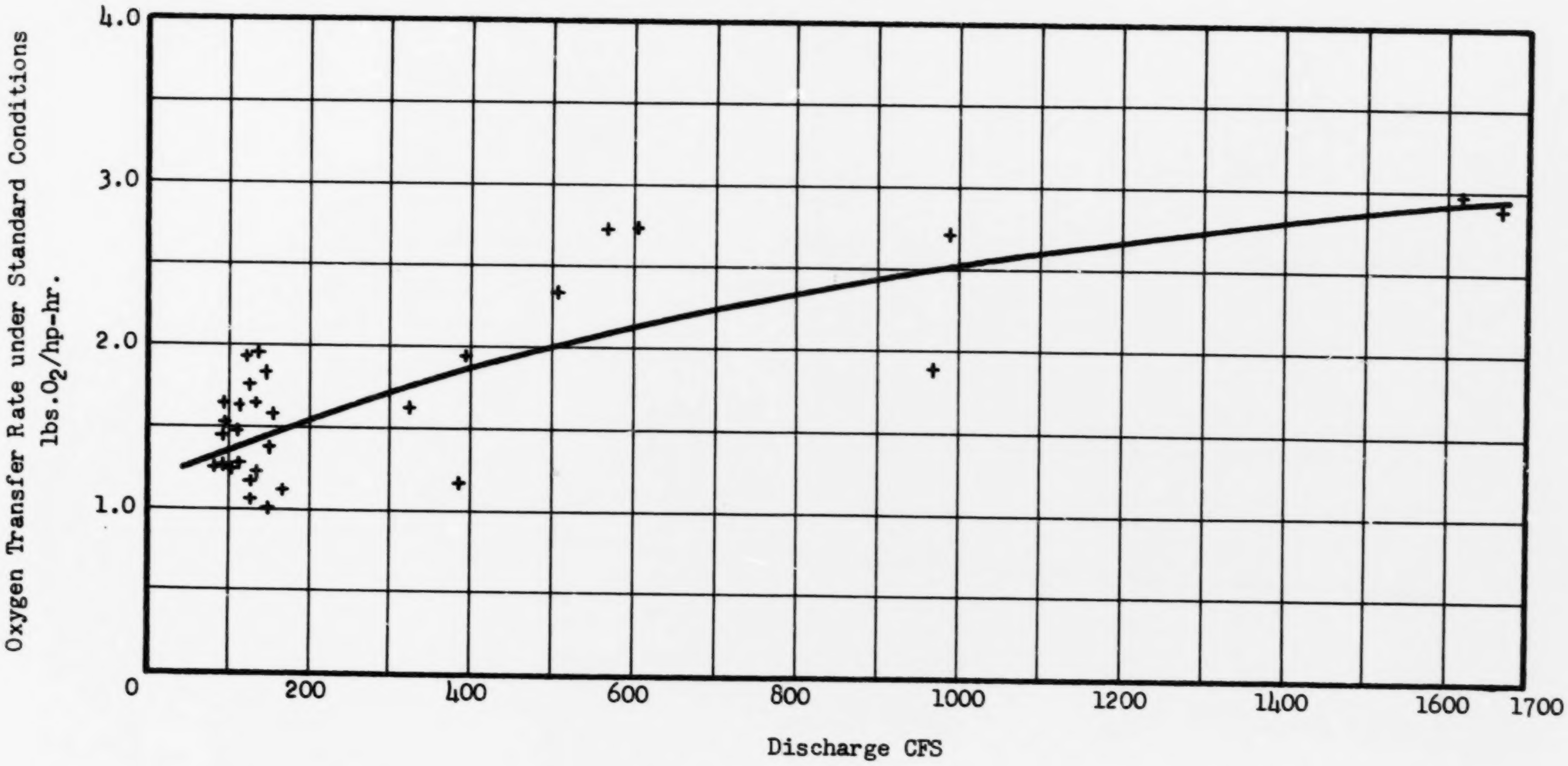


FIG. 4-10

OXYGEN TRANSFER RATE VS FLOW MECHANICAL AERATOR

(Based on upstream D.O. concentration, i.e. $C_m = C_u$)

SAMPLING POINT 200 FT. APART

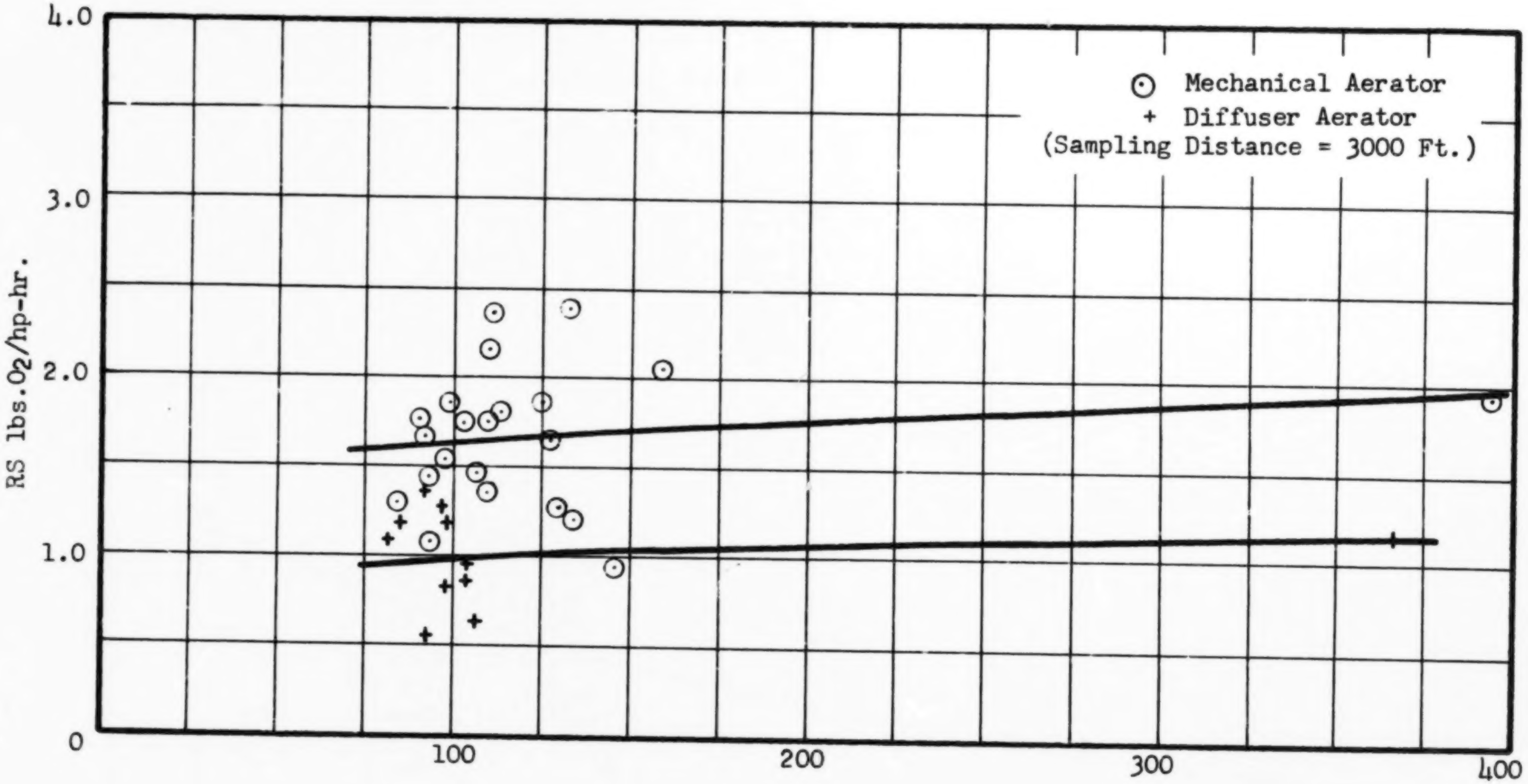


FIG. 4-11

OXYGEN TRANSFER RATE VS FLOW

MECHANICAL AND DIFFUSER AERATORS

the river is assumed to be 100 cfs, the water temperature 25°C. From Figure 4-5 it is found that the oxygen transfer rate under standard conditions, R_S, for Q = 100 cfs averages about 1.9 lb O_2/hp-hr. To convert this to actual conditions, the conversion factor, F, can be found from Figure 4-3 for $C_m = 1/2\ (1.0 + 5.0) = 3.0$ mg/l and T = 25°C, to be 1.71. Therefore

$$R_T = \frac{R_S}{F} = \frac{1.90}{1.71} = 1.11 \text{ lb } O_2/\text{hp-hr} \quad \text{.... Eq. 4-13}$$

Now since

$$R_T = \frac{B\ Q\ (C_d - C_u)}{P} = \frac{0.2246\ Q\ (\Delta C)}{P} \quad \text{.... Eq. 4-14}$$

with known value of R_T, two alternative procedures can be used:

(a) if ΔC is given, as in this case at (5.0-1.0) or 4.0 mg/l, from equation 4-14 the shaft power required, P, can be determined, and thus the size of the aerator needed can be decided. For $\Delta C = 4.0$ mg/l, P = 81 hp. Therefore, an aerator with 81 shaft horsepower is needed.

(b) if the size of the aerator is given, for example, as 67.5 shaft horsepower, then equation 4-14 can be used to compute the oxygen level to be raised by such an aerator. In this case, since the downstream d.o. concentration, C_d, is unknown, a trial-and-error procedure must be applied to determine ΔC if method R_S-#2 is used. For this example we have

$$R_T = \frac{1.90}{F} = \frac{0.2246\ (100)\ (\Delta C)}{67.5}$$

In the above equation both F and ΔC are unknown. By trial-and-error method we can obtain

$$C_d = 4.4 \text{ mg/l}, \ \Delta C = 3.40 \text{ mg/l}$$

Therefore one aerator of this size can only raise the d.o. level to 4.4 mg/l. If no other size equipment is available, two aerators must be used.

To facilitate the computation, method R_S-#1 may be used instead of R_S-#2, because in this case C_m is assumed to be C_u and no trial-and-error procedure is needed to determine C_m. For this example, if $C_m = C_u = 1.0$ mg/l from Figure 4-6 we find that $R_S = 1.35$ lb O_2/hp-hr, and from Figure 4-3 F = 1.31. Therefore

$$R_T = \frac{1.35}{1.31} = 1.03 \text{ lb } O_2\text{/hp-hr, and}$$

$$R_T = \frac{0.2246(100)\ (\Delta C)}{67.5} = 1.03$$

$$\Delta C = 3.10 \text{ mg/l}$$

It is observed that by method R_S-#1, an aerator of the given size can raise the d.o. level to 4.10 mg/l. Two aerators are also needed to meet the given requirement. In this particular example method R_S-#1 gives the same number of aerators required as method R_S-#2 even though the former underestimates the aerator performance, as described earlier. The actual application of these procedures to the design of aeration system for the Passaic River is illustrated in Chapter VI.

ACKNOWLEDGEMENT

The figures and some of the computations for this chapter were made by Mr. William Connerty. The help of Mrs. Marjorie Krespach in preparing the figures is also appreciated.

APPENDIX IV-1

OXYGEN TRANSFER OF AERATORS: THEORETICAL ASPECTS

To date the available techniques for evaluating aerator performance are mostly derived from the classical "two-film" theory proposed by Lewis and Whitman in 1924.(12) They assumed that at the interface between a gas and a turbulent liquid there exist two films, one gas and one liquid. A steady-state transfer across the films is assumed, and the gas must pass through these films by molecular diffusion. Considering a uniform concentration in the liquid and a saturation concentration at the interface, they arrived at the following expression:

$$\frac{dC}{dt} = K_L \frac{A}{V} (C_s - C) = K_L a \, (C_s - C) \qquad \text{.... Eq. 4-15}$$

where

C = concentration of the dissolved gas in the body of the liquid

$\frac{dC}{dt}$ = rate of change of C with respect to time

K_L = the "liquid film coefficient"

A = aera of the interface

V = volume of the liquid

C_s = saturation concentration of gas

a = interfacial area per unit volume of liquid

$K_L a$ = the overall transfer coefficient

Physically the equation suggests that the rate of gas transfer is proportional to the concentration gradient across the film. In the case of oxygen transfer, the term $(C_s - C)$ is referred to as the dissolved oxygen deficit, or the "driving force" of the system. Let $D = C_s - C$, equation 4-15 can be written as

$$\frac{dD}{dt} = -K_L a\, D \qquad \text{.... Eq. 4-16}$$

which integrates to

$$D = D_o\, e^{-K_L at} \qquad \text{.... Eq. 4-17}$$

where D_o is the initial d.o. deficit.

The commonly used unsteady-state method of determining aeration efficiency in tanks or lagoons involves first the computation of the over-all transfer coefficient, $K_L a$. The procedure is fairly simple. For example, water with low oxygen content is aerated and the change in dissolved oxygen content with time is observed. The slope of a semi-logarithmic plot of the dissolved oxygen deficit, D, vs time then yields $K_L a$. For a system of volume V, the total oxygen uptake due to the aerator can be computed from(3)

$$\text{uptake} = (K_L a)\ (D)\ (V) \qquad \text{.... Eq. 4-18}$$

The oxygen transfer efficiency for the aerator under test conditions in terms of pounds of oxygen per horsepower hour is obtained by

$$R_T = \frac{B\ (K_L a)\ (D)\ (V)}{P} \qquad \text{.... Eq. 4-19}$$

in which

R_T = oxygen transfer rate under test conditions

B = unit conversion factor

P = shaft power in horsepower

The oxygen transfer rate defined by equation 4-19 is assumed to completely characterize the performance of an aerator, and is commonly used as a basis for comparing the efficiency of different aerators.

In addition to the unsteady-state technique described above, there are other techniques used to evaluate aeration performance, such as the steady-state and unsteady-state aeration of activated sludge, etc. A general review of these methods can be found, for example, in Eckenfelder and Ford.(6)

The basic assumptions of the two-film theory are that the oxygen concentration is uniform throughout a definite liquid volume, V, and that steady-state transfer exists across the film. In field operations, particularly in the case of stream aeration, such conditions do not exist in the immediate vicinity of the aerators. Consequently the above relationships cannot be applied.

Mathematical relationships have been proposed for the computation of oxygen transfer efficiency of aerators in natural streams.(7)(8)(9) Susag et al.(7) applied an equation of the form of equation 4-1 to their steady-state laboratory channel results. Kaplovsky et al.(9) defined an absorption rate coefficient, which is similar to what Kalinske(8) suggested to be the proper term that characterizes the performance of a localized aerator in natural streams. These coefficients have the dimensions of a flow rate, which is different from the most commonly used expression for aerator performance, namely, pounds of oxygen per horsepower-hour, as described by equation 4-1.

Under certain assumptions, the basic equation describing the distribution of dissolved oxygen in a natural stream may be written as(13)

$$u\frac{\delta C}{\delta t} + \frac{u\delta C}{\delta x} = k_2\ (C_s - C) - k_1 L + \Sigma S \qquad \text{.... Eq. 4-20}$$

in which

u = average stream velocity in the x-direction

k_2 = the "normal" reaeration coefficient

k_1 = the b.o.d. reaction rate coefficient

L = the first stage b.o.d.

S = source and sinks of oxygen, referring to, for example, benthal demand, algae respiration and photosynthesis.

Consider the case that an aerator is located at the mid-point of a river reach of length s. Dissolved oxygen concentrations are measured at a distance of $\frac{s}{2}$ upstream and $\frac{s}{2}$ downstream of the aerator. Assuming the "driving force" for the oxygen transfer due to the aerator is $(C_s - C_m)$, equation 4-20 may be written as

$$\frac{\delta C}{\delta t} + u\frac{\delta C}{\delta x} = K_L a\ (C_s - C_m) + k_2\ (C_s - C) - k_1 L + \Sigma S \qquad \text{.... Eq. 4-21}$$

For small s and short duration of time it is reasonable to assume that the term $K_L a\ (C_s - C_m)$ is predominating compared to the other terms on the right hand side of equation 4-21. In addition, it is assumed that a linear approximation of the longitudinal d.o. variation from upstream to downstream could be adopted with aerator in operation. Equation 4-21 can be simplified to the form

$$\frac{dC}{dt} + \frac{Q(C_d - C_u)}{V} = K_L a\ (C_s - C_m) \qquad \text{.... Eq. 4-22}$$

where

Q = the river flow rate

V = water volume between upstream and downstream sampling points

C_d = downstream d.o. concentration

C_u = upstream d.o. concentration

Under steady-state conditions $\frac{dC}{dt} = 0$, an oxygen transfer coefficient, defined as K, for the aerator can be readily obtained

$$K = \frac{Q\ (C_d - C_u)}{C_s - C_m} \qquad \text{.... Eq. 4-23}$$

It is clear that $K = (K_L a)\ (V)$, and has the dimensions of Q. After proper conversion of units, the coefficient, K*, can be also interpreted as the rate of oxygen added by the aerator per unit "driving force."** Equation 4-23 can be related to equation 4-1 as follows:

$$R_T = \frac{Q\ (C_d - C_u)}{P} = \frac{K\ (C_s - C_m)}{P} \qquad \text{.... Eq. 4-24}$$

From equation 4-8 it is noted that the parameter "K" has been included when converting R_T to R_S. Equation 4-24 was used to compute R_T under steady-state conditions, as mentioned in Chapter IV.

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*This term "K" should be distinguished from symbols K_a, K_d., etc. used later in the test for other parameters.

**The term "$K_L a$" has also been used to describe the relationship 4-23.(6)(14) But then the dimensions became inconsistent with the $K_L a$ in equation 4-15. Therefore the parameter "K" is adopted here.

For unsteady-state conditions, equation 4-22 could be applied to downstream sampling point and could be integrated for certain initial conditions to yield

$$t = \frac{2\ V}{2\ Q + K} \quad \ln \left[\frac{\frac{K}{V}\ (C_s - C_u)}{\frac{K}{2V}\ (2C_s - C_u - C_d) - \frac{Q}{V}\ (C_d - C_u)} \right]$$

.... Eq. 4-25

To obtain equation 4-25, it is assumed that $C_m = \frac{1}{2}\ (C_d + C_u)$ and that when $t = 0$, $C_d = C_u$.

Although the unsteady-state equation could theoretically provide an independent check on the aeration efficiency,(8) it is not recommended for general use in river aeration practices because of the limitations in deriving the equation, and the inherent difficulty in estimating accurately the water volume, V.

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Chapter V

RIVER AERATION SYSTEMS AND THEIR COSTS

Basic Assumptions

Induced river aeration is considered in these studies as a supplement to secondary treatment of wastes, and an alternative to advanced waste treatment. It is assumed that a basin water quality plan will be prepared, based upon a systems analysis, as a result of which aeration stations will be located along the river in the most advantageous manner. It is assumed that the river will be sufficiently small that the effects will be dispersed rapidly across its width, and that the aeration stations will be fixed in location. The final assumption is that an organization will exist to monitor water quality and operate the aeration stations on a scientific basis in order to utilize their capacity effectively.

Costs

In order to broaden the applicability of the cost data, estimates were obtained for construction, operation and maintenance of a range of hypothetical alternative systems described by number and size of units. In Chapter VII, these results are applied to obtain costs of the various systems found to be optimum to meet various stated requirements.

Cost estimates were prepared by the engineering firm Hazen and Sawyer, based upon the following.

1. Aerators and components were priced at 1969 prices using equipment commercially available on a competitive basis. The diffuser aerator blower was sized for 10 foot water depth. (The diffuser aerator actually used had excess blower capacity,

in view of the planned use later in deeper water on the Delaware.)

2. The costs of aeration installation, related electrical equipment, and shelter were based upon experience on the Passaic River. The electric service costs were estimated for a distance of about 500 feet from the power source. Estimates were prepared for six different cases. Each case referred to a different type or size of aerator, and each was estimated for a range of three different numbers of units. The six cases were as follows:

1. For 75 hp electric drive mechanical aerators. Estimated for systems consisting of 3, 6 and 9 aerators, respectively.
2. For 50 hp electric drive mechanical aerator, estimated for systems of 4, 8 and 12 aerators, respectively.
3. For 80 hp electric drive diffuser aerator, basic design, 4, 8 and 12 aerators, respectively.
4. Same as 3 for diesel drive.
5. For 80 hp electric drive diffuser aerator, with a seven manifold low-turbulence design, estimated for 4, 8 and 12 aerators, respectively.
6. Same as 5 for diesel drive.

It was originally intended to prepare alternative estimates also for mechanical aerators mounted on rafts with diesel drive, which would be economical for locations where necessary electric energy was not available in the vicinity. Such aerators are not commercially available, and some of their components would have to be specially designed, so that comparable cost estimates were not prepared.

The selection of these six cases was designed to provide maximum coverage of potential aeration systems in a variety of rivers. Aeration systems obviously will be proportionately more economical to install and to operate with a larger number of units. The range of numbers of units of each type is planned to show this relationship. It was, or course, necessary to make independent estimates of costs of mechanical and of diffuser aeration systems, as one of the basic purposes of the project. Moreover, it was desired to provide costs of a special design of low-turbulence aerator installation for use in the event that the turbulence created by normal aerators should prove objectionable for any reason.

As regards size, the 75 hp mechanical aerator actually tested is near the upper limit of commercially available equipment. It was brought out in preliminary discussions (as illustrated in more detail in the following chapter), that properly spaced small aerators will always provide a larger oxygen uptake than can be achieved by larger aerators of the same total capacity, and the same efficiency. This was the reason for the comparison of costs of optimal systems using 50 and 75 hp aerators, respectively. The 50 hp unit estimates also provide a cost basis for induced aeration in areas where there may be practical reasons why 75 hp units are too large, such as insufficient size of stream.

Capital recovery costs

The capital recovery costs were estimated in accordance with interest rate principles previously developed by one of the investigators. (1) The average yield of Federal long term bonds in May 1969 was 5.88%. (2) In principle, this rate would be reduced by a currency depreciation factor, taken as at least 1.0%; but there is also a considerable risk element, which is not allowed for in Federal bond rates, which would require an increase in the riskless rate. Therefore, it was decided to use a basic interest rate of 6%. For a life of 20 years, this gives a capital recovery rate (interest plus amortization) of 8.72%.

Operation and Maintenance

Estimates were based upon the assumption that each unit is a separate installation, located a significant distance from the next unit, but close enough that maintenance personnel can drive readily from one to another. It was assumed that the mechanical surface aerators would be removed each winter from the river for about four months for overhaul and protection from ice, and that the diffuser manifolds would be removed once every four years for cleaning and repair.

The fuel and electric energy consumption was based upon the assumption that during an average year, the aerators would be in continuous (24 hour -7 day) operation for 3 months, in half-time operation for 5 months, and inactive for 4 months. Actual schedules would, of course, depend upon seasonal flow duration and water quality characteristics of the river in question.

Personnel costs included provision for a daily visit to each site, with one man to inspect, operate, and maintain a number of installations, and with an additional maintenance allowance of 3% annually of total material costs for more important repairs. Electric power costs were based upon the rate schedule for large users of the Public Service Gas and Electric Co. of N.J. Diesel fuel estimates were based upon #2 diesel fuel oil.

Total System Costs

The cost estimates made on the above basis are summarized in Tables 5-1 to 5-3 below.

TABLE 5 - 1

COSTS

MECHANICAL SURFACE AERATION SYSTEMS

CASE NUMBERS	I A	1 B	1 C	II A	II B	II C
No. UNITS	3	6	9	4	8	12
AERATOR	Electric 75 h.p.	Electric 75 h.p.	Electric 75 h.p.	Electric 50 h.p.	Electric 50 h.p.	Electric 50 h.p.
CONSTRUCTION COSTS						
Mechanical aerator (material only)						
Installation of aerator						
Electrical equipment and installation						
Excavation						
Shelter						
Miscellaneous						
Subtotal	$110,000	$219,000	$326,000	$133,000	$255,000	$369,000
Contingencies and Engineering @ 20%	22,000	44,000	65,000	27,000	51,000	74,000
Total	$132,000	$263,000	$391,000	$160,000	$306,000	$443,000
ANNUAL OPERATION AND MAINTENANCE COSTS						
Personnel						
Electric power						
Removal of aerator and re-installation						
Maintenance						
Miscellaneous						
Total O&M	$30,000	$50,000	$68,000	$30,000	$50,000	$70,000
Capital recovery costs 6% - 20 yrs	11,500	22,900	34,000	13,900	26,700	38,600
TOTAL ANNUAL COSTS	$41,500	$72,900	$102,000	$43,900	$76,700	$108,600

TABLE 5 - 2

COSTS

DIFFUSION AERATION SYSTEMS

BASIC DESIGN*

CASE NUMBER	IIIA	IIIB	IIIC	IV A	IV B	IV C
No. UNITS	4	8	12	4	8	12
DRIVE	Electric	Electric	Electric	Diesel	Diesel	Diesel
CONSTRUCTION COSTS						
Blower and drive unit						
Diffusion system						
Electrical equipment and installation						
Excavation						
Shelter						
Air diffuser nozzles						
Miscellaneous						
Subtotal	$170,000	$340,000	$510,000	$168,000	$335,000	$500,000
Contingencies and Engineering @ 20%	34,000	68,000	102,000	34,000	67,000	100,000
TOTAL	$204,000	$408,000	$612,000	$202,000	$402,000	$600,000
ANNUAL OPERATION AND MAINTENANCE COSTS						
Personnel						
Electric power						
Removal of aeration piping						
Fuel oil for diesel unit						
Maintenance						
Miscellaneous						
TOTAL O&M	$37,000	$63,000	$88,000	$37,000	$68,000	$98,000
Capital recovery costs 6% - 25 yrs.	17,800	35,500	53,400	17,600	35,100	52,300
TOTAL ANNUAL COSTS	$54,800	$98,500	$141,400	$54,600	$103,100	$150,300

* 80 h.p., two manifolds, 12 holes open on diffusers.

TABLE 5 - 3

COSTS

DIFFUSION AERATION SYSTEMS

SPECIAL DESIGN
LOW TURBULENCE*

CASE NUMBER	V A	V B	V C	VI A	VI B	VI C
No. UNITS	4	8	12	4	8	12
DRIVE	Electric	Electric	Electric	Diesel	Diesel	Diesel
CONSTRUCTION COSTS						
Blower and drive unit						
Diffusion system						
Electrical equipment and installation						
Excavation						
Shelter						
Air diffuser nozzles						
Miscellaneous						
Subtotal	$205,000	$414,000	$616,000	$203,000	$409,000	$606,000
Contingencies and Engineering @ 20%	45,000	90,000	134,000	44,000	89,000	132,000
TOTAL	$250,000	$504,000	$750,000	$247,000	$498,000	$738,000
ANNUAL OPERATION AND MAINTENANCE COSTS						
Personnel						
Electric power						
Removal of aeration piping						
Fuel oil for diesel unit						
Maintenance						
Miscellaneous						
TOTAL O & M	$38,500	$66,000	$92,500	$38,500	$71,000	$102,500
Capital recovery costs 6% - 25 yrs	$21,800	$43,900	$65,400	$21,700	$43,500	$64,300
TOTAL ANNUAL COSTS	$60,300	$109,900	$157,900	$60,200	$114,500	$166,800

* 80 h.p., seven manifolds, 4 holes open on diffusers.

Comparison Between Diffuser and Mechanical Aerators

An exactly comparable statistical comparison between the two types was not necessary, in view of the clear cut results achieved. As is shown in Chapter IV and illustrated on Fig 4-7, the transfer rate of the diffuser aerator was only about 60% that of the mechanical aerator under comparable conditions.

The diffuser aerators specified in cost estimates have somewhat greater output capacity than the mechanical aerators. With a rated output of 80 hp, they can be operated for considerable periods at 90 hp. However, for reliability and long service, they are considered as capable of rated capacity only.

The 75 hp mechanical aerator, in river use, showed a tendency to overload initially, and then diminished gradually in output over a period of hours, sometimes to as low as 60 hp. However, it would not be difficult to devise some control device, so that the equipment could develop its rated capacity under all normal operating conditions. Therefore, a cost comparison can be made on the basis of rated capacity in each case.

From Table 5-1 above, nine 75 hp mechanical aerators with a total rated capacity of 675 hp would have total annual costs of $102,000 or $151 per annual hp of rated capacity. From Table 5-2, twelve electric drive diffuser aerators rated at 80 hp each or a total of 960 hp, would have total annual costs of $141,400 annually, or $147 per annual hp of rated capacity. Since the unit transfer rate of the diffuser aerator is so much lower, and the costs per hp-hr. almost the same, it is evident that, based upon the test results, the mechanical aerator would be the more economical by a wide margin.

There would be certain intangibles in favor of the diffuser aerators, particularly the lesser interference with boating, and the fact that the mechanical aerators must be serviced to remove heavy drift, and to adjust the cables or remove the aerators in time of flood. However, the cost advantage seems too great for further consideration of this type of diffuser, at least on such shallow streams. The systems analysis of the next chapter was prepared on the basis of mechanical aerators only.

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Chapter VI

SIMULATION STUDIES AND SUB-OPTIMAL AERATION DESIGN CRITERIA

Introduction

In the previous chapters, hydrological and ecological field data, collected over a two-year period on the Pine Brook - Two Bridges reach of the Passaic River, are presented. Oxygen uptake rates for the mechanical and diffuser aerators are given as observed and as reduced to standard conditions. Laboratory and field measurements of the L_o, N_o, $\overline{P-R}$, B, K_a, and C_s parameters are correlated with mean stream flow rate and temperature. The stage is now set to synthesize the data in an attempt to formulate a rational procedure or strategy for determining the number, hp size, and longitudinal spacing of mechanical surface aerators required to meet certain stream dissolved oxygen (d.o.) quality standards under various flow conditions and BOD loads between Pine Brook and Two Bridges, and Little Falls and Dundee Dam, respectively.

As a first step in developing a viable design strategy, it is necessary to derive a mathematical model for the Passaic River with parameter values determined from actual field measurements of stream d.o. and BOD. The model must be applicable in a general way to both the Little Falls and Pine Brook reaches. Since most of the data collected pertain to the Pine Brook section, some degree of extrapolation and compromising must be made in order to apply the model to other sections of the stream. With a mathematical model available, it is possible to computerize the mechanics involved in obtaining simulated aeration station designs.

Of the two types of computers which were available for the computational work (i.e. the IBM 360/67 digital computer and the E.A.I. TR-48 analog computer), it was decided that the analog computer would be preferable. The kinds of design calculations required for the project necessitated the use of a real time capability for displaying the results quickly and accurately in both oscilloscope and graphical forms. The curve fitting strategy used required numerous trial parameter changes followed by on-the-spot evaluation and reparameterization. The real time capability of the analog computer proved to be indispensible during this phase of the simulation.

Mathematical Model of the Passaic River

The basic factors affecting stream d.o. in the Passaic River are described in Chapter III. In differential equation form, these factors are related to one another according to

$$\frac{dL}{d\tau} = - K_r L \; ; \qquad L\,(0) = L_o \qquad \text{... Eq. 6-1}$$

$$\frac{dN}{d\tau} = - K_n N \; ; \qquad N\,(0) = N_o \qquad \text{... Eq. 6-2}$$

$$\frac{dC}{d\tau} = K_a(C_s - C) - K_r L + \overline{P-R} - B; \; C(0) = C_o \qquad \text{... Eq. 6-3}$$

The integrated form of these equations was given previously by equation 3-2 and is repeated here for reference. (Note that $K_d = K_r$.)

$$D_\tau = \frac{K_r L_o}{K_a - K_r} \left(e^{-K_r \tau} - e^{-K_a \tau}\right) + \frac{K_N N_o}{K_a - K_N} \left(e^{-K_N \tau} - e^{-K_a \tau}\right)$$

$$+ D_o\left(e^{-K_a \tau}\right) + \frac{B}{K_a} \left(1 - e^{-K_a \tau}\right) - \frac{\overline{P-R}}{K_a} \left(1 - e^{-K_a \tau}\right) \qquad \text{... Eq. 6-4}$$

Equations 6-1 to 6-3 constitute the plug-flow, deterministic, mathematical model for the Passaic River. Even though the Passaic River d.o. and BOD are changing with the time-of-day, it is still possible to approximate the system response by a steady-state approach. This tactic is commonly used throughout the field and is well documented by Dobbins (1964) (1) and O'Connor (1966). (2)

The system equations 6-1 to 6-3 are fully linear in d.o. and BOD. This feature allows the principle of superposition to be used in dealing with the effects of introducing staged aerators at the boundary or initial points in the various segments of the river when the aerators are turned on. The effects of ΔC on the downstream d.o. values enter by way of the inlet or initial conditions. For example, integration of equation 6-3 for the case where a ΔC has been added at $\tau = 0$ is effected by merely changing $C(0) = C_o$ in equation (6-3) to $C(0) = C_o + \Delta C$. This is the principle of superposition alluded to above. The value of ΔC is calculated according to the R_s #1 or R_s #2 methods previously discussed in Chapter IV.

The Curve-Fitting Strategy

The present state-of-the-art in curve-fitting procedures does not allow for perfect delineation between the various parameters in the Passaic River model. For example, in the context of a nonlinear regression curve-fitting procedure, all of the parameters except $\overline{P-R}$ and B are coupled together in a nonlinear fashion. This situation all too often leads to polymodality and multiple parameter sets which are difficult to arbitrate. It is preferable in the long run to substitute whenever possible a dynamic curve-fitting routine based on the differential equation form of the model using an analog computer. Arbitrating between multiple parameter sets on the analog computer is a far less cumbersome task for the practitioner in the field. Dynamic simulation procedures on the analog computer can and do provide a systematic method for surveying various parameter sets selectively in a wide range of combinations.

The analog computer program for the parameter search routine is shown in Figures 6-1a and the computer itself in 6-1b. Basically, a rotational reparameterization procedure was used in conjunction with a ranking or hierarchy of the parameters. The hierarchy of the parameters was determined according to their ability to be estimated by an independent, reliable empirical formula or field observations. In Chapter III, it has been shown how certain parameters could be estimated by empirical formula or correlation. For example, C_s was determined rather accurately by equation 3-3 as a function of temperature and is therefore considered to be highly deterministic. The ranking of the parameters, starting with the most unreliable ones first is as follows:

(i) K_r, K_N, B, and K_a

(ii) L_o and N_o

(iii) $\overline{P-R}$

(iv) $\bar{u}$ (mean stream velocity)

(v) C_o

(vi) C_s

(vii) T

The value for C_o is determined directly from the field measurements using the Delta DO meter and Winkler methods. The value for $\bar{u}$ is determined using volumetric flow measurements at Pine Brook in conjunction with the following empirical formula (See App. I-1):

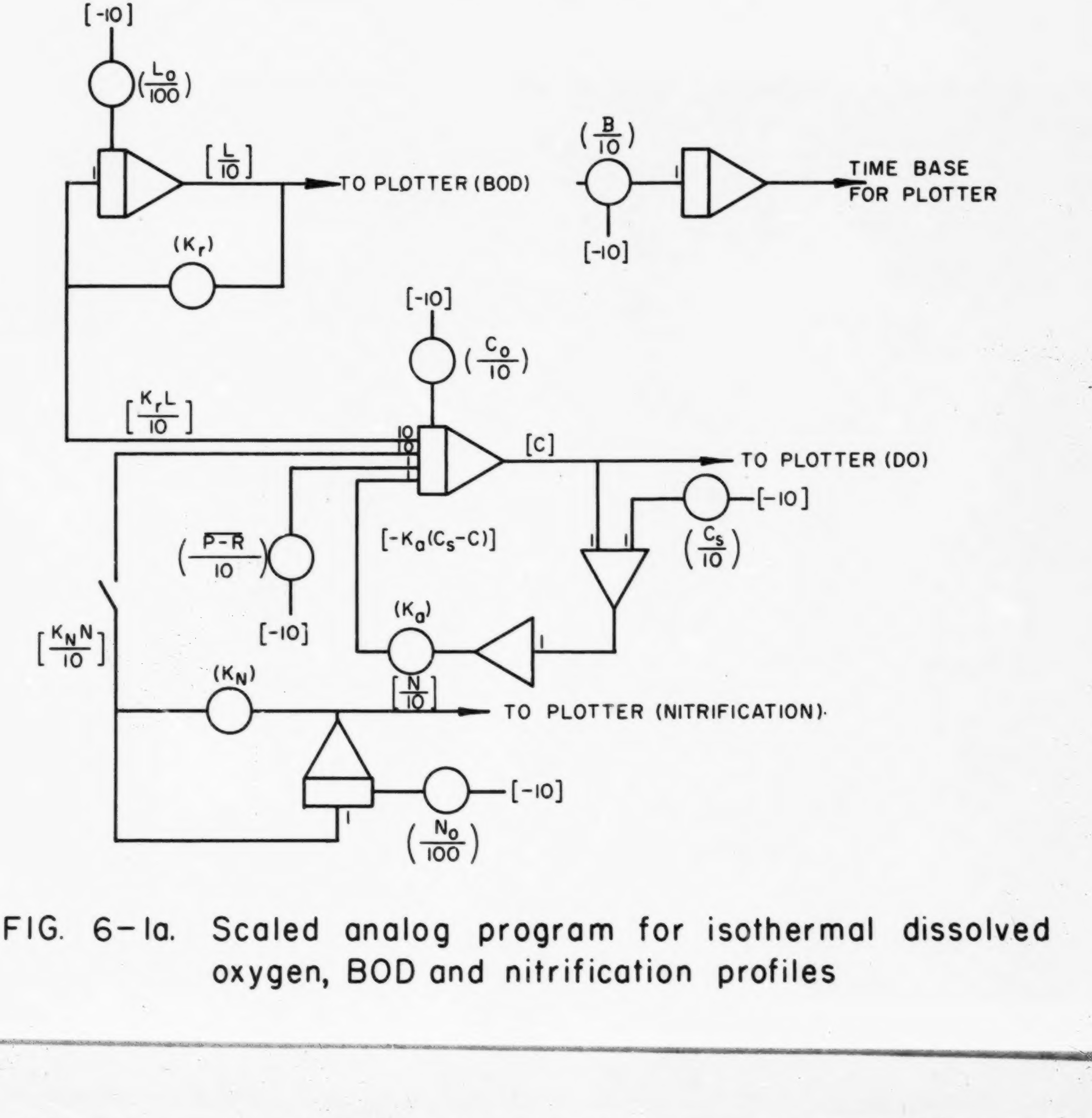
[-10]
$(\frac{L_o}{100})$
$[\frac{L}{10}]$
TO PLOTTER (BOD)
(K_r)
$(\frac{B}{10})$
[-10]
TIME BASE FOR PLOTTER
[-10]
$(\frac{C_o}{10})$
$[\frac{K_r L}{10}]$
10
10
[C]
TO PLOTTER (DO)
[-10]
$(\frac{C_s}{10})$
$(\frac{\overline{P-R}}{10})$
[-10]
$[-K_a(C_s-C)]$
(K_a)
$[\frac{K_N N}{10}]$
(K_N)
$[\frac{N}{10}]$
TO PLOTTER (NITRIFICATION)
[-10]
$(\frac{N_o}{100})$

FIG. 6-1a. Scaled analog program for isothermal dissolved oxygen, BOD and nitrification profiles

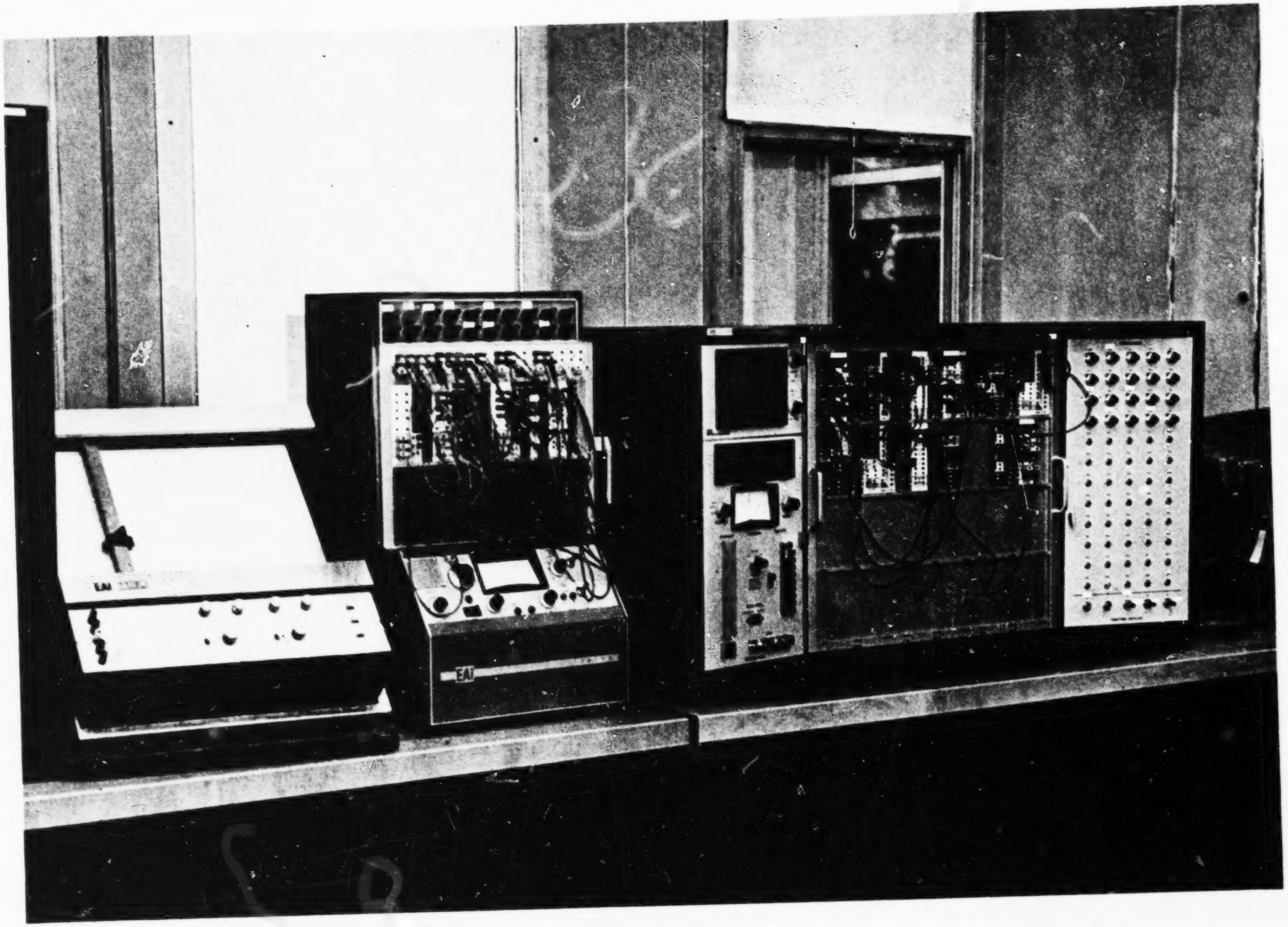

FIGURE 6-1b

THE E.A.I. TR-48 ANALOG COMPUTER

$\bar{u} = 0.0199\ Q^{0.524}$

$\bar{u}$ = average velocity between Pine Brook and Two Bridges in ft./sec.

Q = volumetric flow at Pine Brook in cfs.

It was decided that on the analog computer the parameters K_r, K_N, L_o, and N_o should receive maximum parameter rotation and variation. The other parameters, except for B and K_a, were fixed according to the empirical formula estimates and correlations from Chapter III. The number of data points surrounding the estimates of B was felt to be too few in number to warrant consideration of B in the model. We therefore set B = 0. The decision was made not to rely on any of the known formulas for quantifying K_a. A brief look again at some of the available correlations shown graphically in Figure 3-2 can help explain this decision. It is observed that the formulas do not agree as to the magnitude of K_a in the flow range of interest in the Passaic River (i.e. at flows between 50-400 cfs). For example, at 400 cfs, K_a varies from 0.166 to 0.570 days^{-1}. The only usable information from the K_a correlations lies in the fact that K_a decreases as flow increases, which is an important factor to be dealt with in the simulated designs. Since K_a is quite possibly the least-known parameter in the group, it was decided to adopt the following heuristically determined values for K_a: K_a = 0.121 at 370 cfs, 0.184 at 120 cfs, 0.201 at 93 cfs, 0.218 at 74 cfs, and 0.260 at 54 cfs. No temperature correction is attempted. These values for K_a, although not derivable from any experimental data, do follow the trend with increasing flow as demanded from theory. This set of K_a parameters was deliberately chosen so as to underestimate their magnitude in order to further guarantee a conservative overall aeration station design during the simulation runs.

In Figure 6-2, there is shown a representative curve-fitting strategy for a situation where the aerators are in the "off" operate mode, representing conditions on June 17, 1968. The data points shown correspond to the measured values of d.o. by the d.o. meter and Winkler methods. The d.o. data points show a characteristic d.o. sag profile. The minimum point occurs at approximately one mile from the Pine Brook station. As a first step in developing dynamic curve-fitting estimates of K_r, K_N, L_o, and N_o in the system model, it was decided to fix K_r at 0.2 days^{-1} based on the lowest value of K_d in Table 3-9. The value of K_d was set equal to K_r in the simulation as explained in Chapter III. The ultimate value of carbonaceous BOD from Table 3-9 indicates that the level at 110 cfs discharge varies from 6 to 15 mg/l. This range of values permits a restriction to be placed on L_o during the simulation runs. The values shown for C_s and $\overline{P-R}$ were determined using the empirical formulas and correlations of Chapter III.

The simulated d.o. response curves for various combinations of parameters are compared to the actual d.o. output data

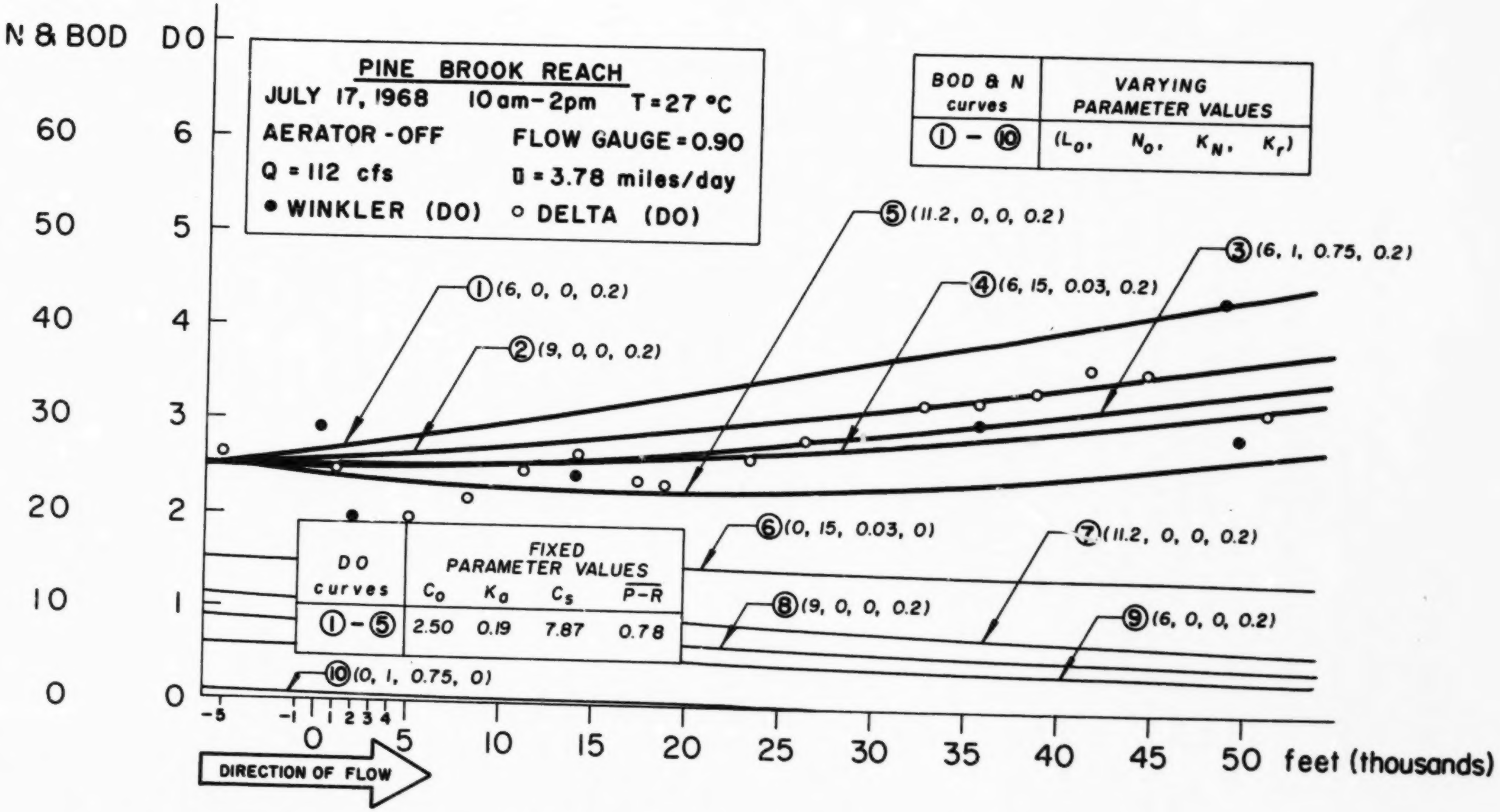

FIG. 6-2. Simulated plots of DO, BOD and nitrification for representative sets of parameters.

from stream measurements. It is not difficult to sense that more than one parameter set can fit the data equally well. This anomaly prevents a unique quantification of K_N, N_o, and L_o. Since measured values of $L_5(\tau)$ were available, it was possible to roughly fix L(0) at the value $L_5(0)$. This approximation provides a lower bound for L(0). In actuality, the observed BOD values probably contain some N_o, which would have to be subtracted out in any event, if it were known or easily measurable. (See Chapter III.) Furthermore, the relationship between laboratory tests and what actually happens in the stream is uncertain. The introduction of K_N and N_o into the system model is complicated by the fact that the nitrification reaction is usually delayed in time relative to the carbonaceous reaction. It was assumed that the delay time was zero throughout the simulation as a simulation convenience. The magnitude of N_o along with K_N derived from the work in Chapter III serves as a rough guideline in the parameter search routine. The nitrification term has been shown to be helpful, but not absolutely essential in simulating the d.o. response curve. For example, curve #2 in Figure 6-2, which is as good a fit as curves #3 and #4, ignores nitrification and favors a larger L_o.

In Figure 6-3, there is shown a simulation strategy for a situation corresponding to a day where the Pine Brook aerator was in the "on" operate mode on July 16, 1968. The ecological and hydrological conditions on the following day roughly corresponded to the hydrological conditions on the previous day (Fig. 6-3).*

The Accelerated Nitrification Phenomenon

The most significant result to be observed is the fact that when the aerator is "on", the d.o. sag curve dips rather fast, especially in the first two or three miles downstream of the aerator. After several parameter set simulation runs, it was concluded that if the L_o and K_r values from the ensuing day (Figure 6-2) were used, then there was a simulation condition which was satisfied by an increase in the K_N parameter from its nominal

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* It should be observed that the aerator was turned on approximately two days before the test was made on July 16, 1968. With a prevailing mean stream velocity of 3.77 miles/day, the nearly steady-state effects of the aerator were felt over a distance of 7.54 miles. In Fig. 6-3, this distance is approximately at the 40,000-foot mark. The curve-fitting of d.o. data points beyond 40,000 feet were treated with less weight than the ones preceding the 40,000 foot mark. Curve #4 in Fig. 6-3 fits the data in the 40,000 foot reach satisfactorily. On July 17, 1968, which represents the test day when the aerator was "off", a period of approximately ½ day elapsed before d.o. data were taken. This represents a distance of approximately 10,000 feet in Fig. 6-2. In the d.o. curve-fitting routine, the data points preceding the 10,000 foot mark were treated with more weight than the ones beyond this distance.

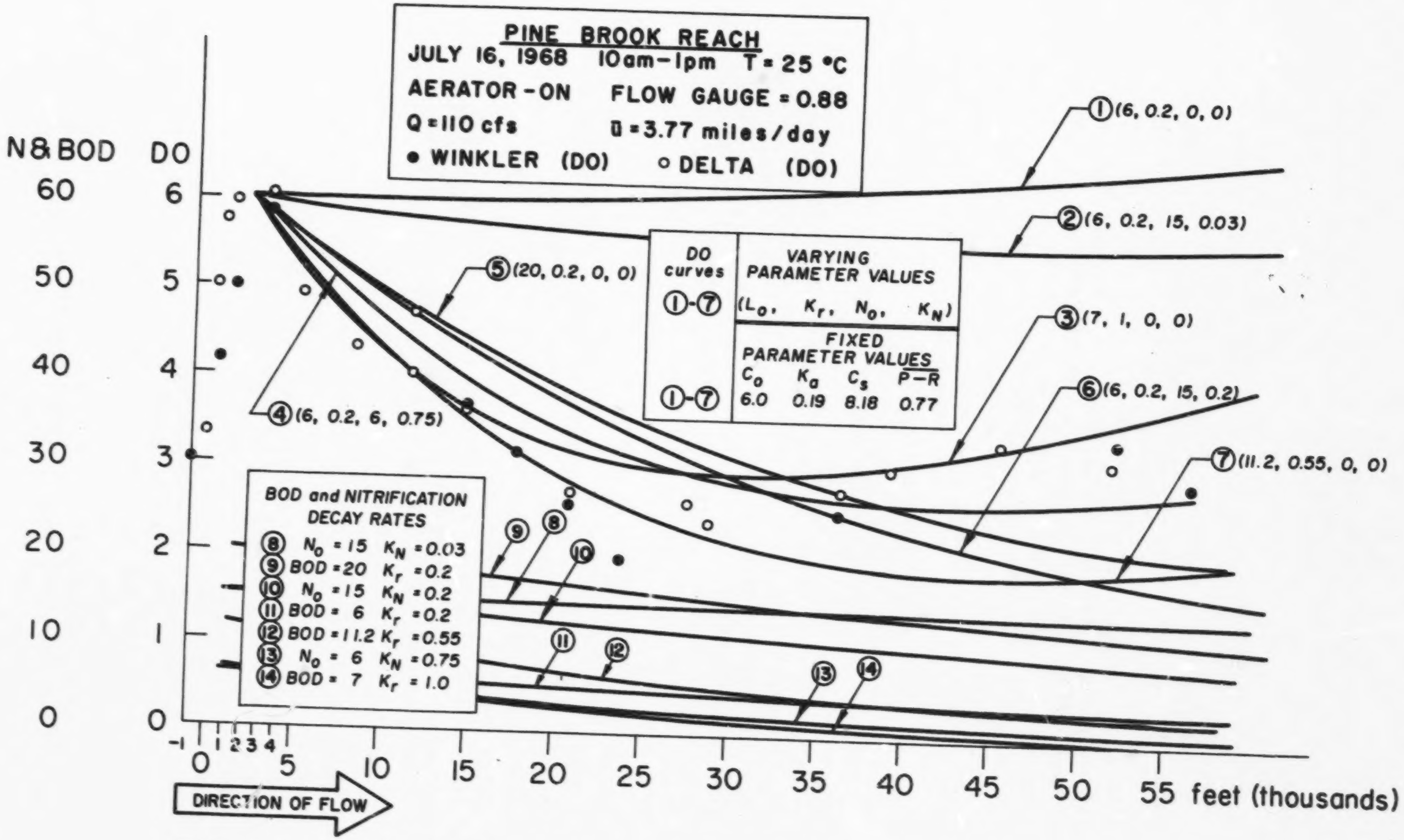

FIG. 6-3. Simulated plots of DO, BOD and nitrification for representative sets of parameters.

value of 0.03 days^{-1} when the aerator was "off" to 0.75 days^{-1} when the aerator was "on". Apparently the aerator triggers some mechanism for accelerated deoxygenation, which persists for several miles downstream. It is possible that the value of K_r could have increased, but in order for this to have happened (curve #3 in Figure 6-3), the nitrification term in the system model would have had to be lowered. Notice that curve #2 in Figure 6-3 does not fit the d.o. response data well at all, whereas on the ensuing day, this parameter set was a good set from the standpoint of d.o. curve-fitting. The laboratory meaurements of the nitrification effects given in Chapter III definitely show that potential nitrification is present to a considerable degree.

The observation that d.o. decreases rapidly below the aerator, as shown in Figure 6-3, was not an isolated finding. Over the two-year period of tests on the Passaic River, numerous other similar results were obtained. Figures 6-3 and 6-2 represent a typical situation that shows this rather dramatic result. The explanation of this phenomenon has not as yet been fully worked out. The hypothesis is advanced that in this stream at Pine Brook certain preconditions for nitrogenous BOD had been fulfilled, so that the turbulence and increased dissolved oxygen of the aeration process triggered a substantial nitrogenous BOD reaction. Assuming that the nitrification demand is accelerated to a significant degree, this means the stream can act like a treatment vessel, in a manner analogous to carbonaceous BOD treatment in an activated reactor. Therefore, it was decided to use parameter sets similar to the one used to obtain curve #4 in Figure 6-3. In this way the potential effects of triggering an accelerated nitrification mechanism on aerator station design and economics can be studied.

To summarize the results of the curve-fitting strategy, it is observed that although numerous parameter sets can be made to fit most any d.o. response situation, there is substantial evidence supporting the use of an accelerated nitrification effect when the aerators are in the "on" operate mode. In all probability, there may be also an enhanced carbonaceous effect, but the evidence supports more strongly the enhanced nitrification hypothesis. Of the two possible ways in which an enhanced nitrification effect could be triggered or demonstrated mathematically (i.e. a large N_o coupled with a small K_N or a large K_N coupled with a modest N_o) the one favoring a large K_N effect seems to yield the better curve-fitting in d.o., as shown in Figure 6-3. If the mathematical model is well set with respect to the direction the various factors may take with temperature change and under different flow conditions, then the simulation model will still supply the designer with useful information, even though the value of any parameter is not uniquely determined. For example, whether one needs more or less aerators as flow increases will certainly be determinable in the model chosen, because the various parameters are correlated, with internal consistency, with flow rate.

Systems of Staged, In-stream Mechanical Surface Aerators

In Chapter V a preliminary cost analysis revealed that single stage aeration using mechanical aerators for medium sized streams (Passaic River size) is significantly more economical than using diffuser aerators. In this section only mechanical surface aerators of the 50 and 75 hp size are investigated.

Figure 6-4 shows the number of mechanical surface aerators required to achieve either 4 or 5 mg/l d.o. starting from an initial value of d.o. of 1.6 mg/l.* Figure 6-4 represents a design under a specified flow of 370 cfs. This flow rate represents only a 12% probability of exceedance during the summer period, as indicated in Appendix I-1. At this flow rate, according to the correlation developed for BOD_5 versus flow (Chap. I), the value of L_5 was approximated to be 7.25 mg/l. In essence, the recorded value of L_{o5} reflects an unknown amount of nitrification. As mentioned previously, it was found necessary, in the absence of a knowledge of the magnitude of the nitrification term, to simply set L_5 equal to L (ultimate) as a conservative estimate, net of nitrification. This practice was followed throughout the simulation study. In Figure 6-4 the parameter set omits the nitrification reaction. The effects of including a nitrification term is discussed below.

The C_s, K_a, and $\overline{P-R}$ parameters were quantified according to the correlations of Chapter III. The value for K_r was fixed at 0.2 days^{-1} as representing carbonaceous BOD removal rate in the Passaic River. The oxygen uptake rates used corresponded to the values recorded in Chapter IV. The method used to calculate ΔC across each aerator was based on method R_s#1, which is discussed in Chapter IV. The distance between aerators was arbitrarily set at 0.7 miles. To achieve a value of 5 mg/l of d.o. using 75 hp aerators would, according to the design strategy shown in Figure 6-4, require 5 aerators, whereas to achieve a desired value of d.o. of 4 mg/l would require only 3 aerators. In Figure 6-5 the design specification based on a ΔC calculated according to the R_s#2 method, which was discussed in Chapter IV is shown. The totality of the results are presented in Table 6-1. It is apparent that aerator design specifications based on R_s#2 always give the same or a smaller number of aerators to achieve the objective. Because of the close agreement between the two methods, R_s#1 method was adopted for the ensuing designs because of its simplicity.

It is interesting to note that as the flow increases, the number of aerators required to achieve the objective also increases, despite the decrease in BOD concentration. This is a

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* The situation in which a stream would be allowed to develop d.o. conditions down to the 1.6 mg/l level prior to river aeration being employed cannot be considered typical.

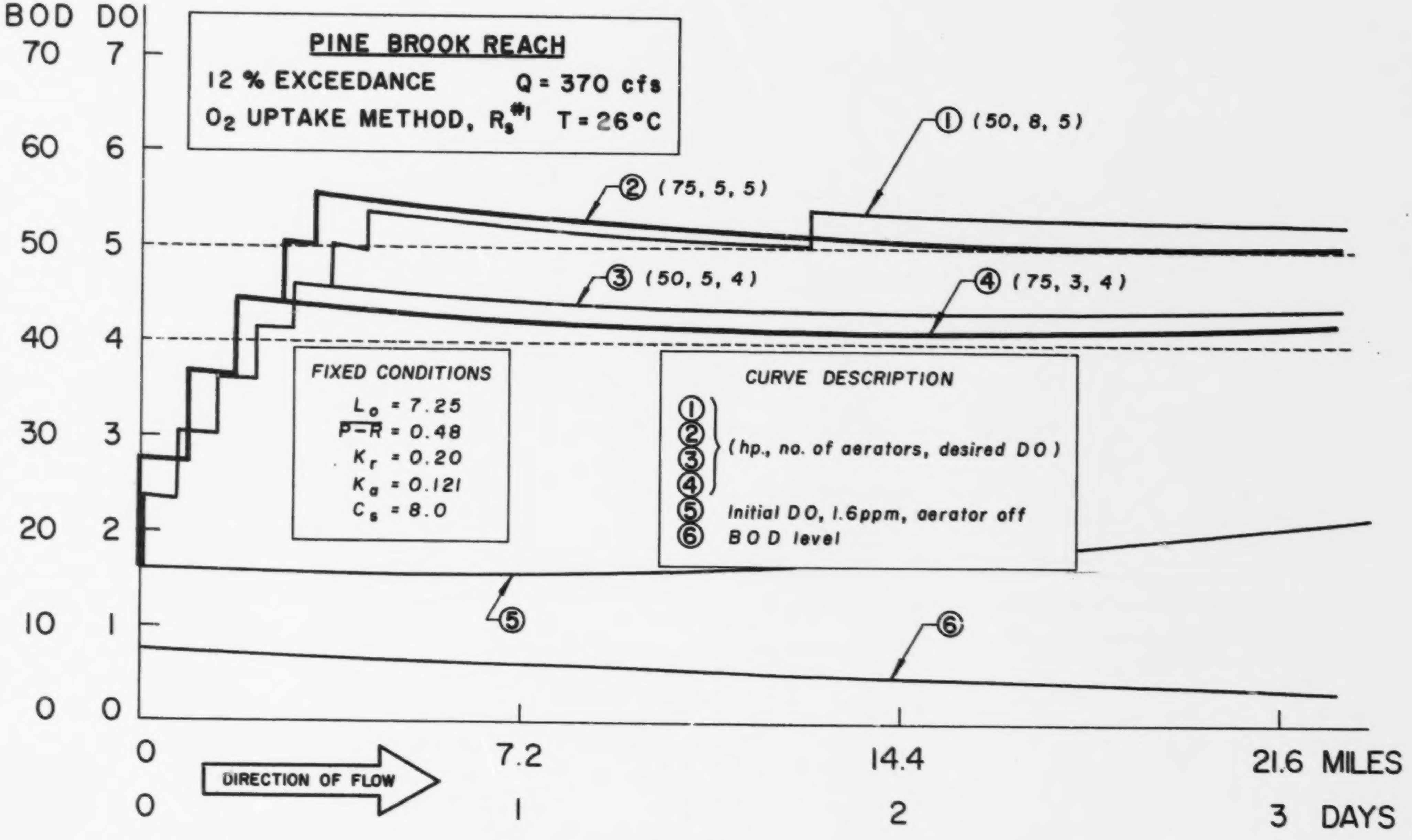

FIG. 6-4. Achievement of DO level (relates to table 6-1)

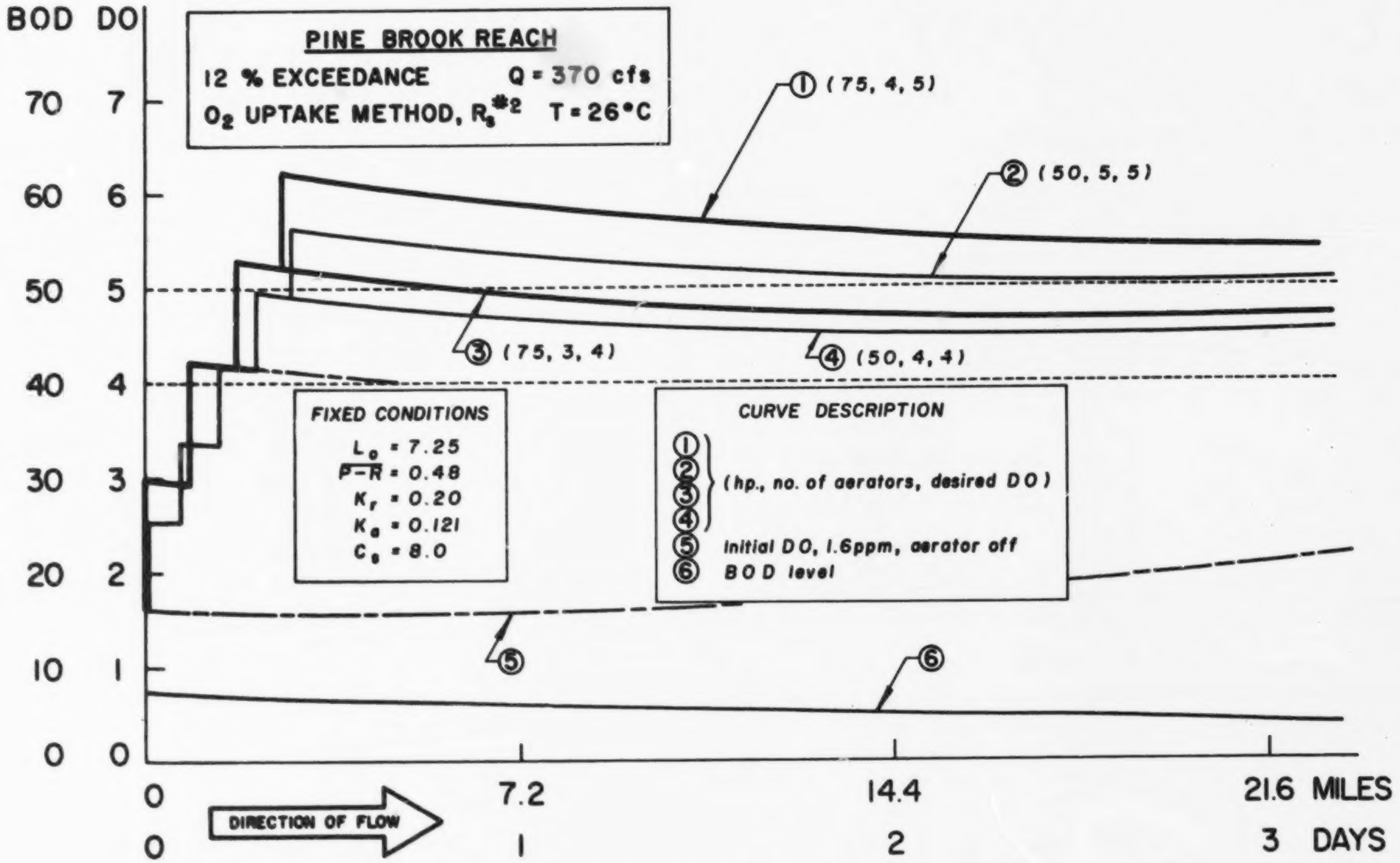

FIG. 6-5. Achievement of DO level (relates to table 6-1)

TABLE 6-1

AERATOR STATION DESIGN SPECIFICATIONS REQUIRED TO ACHIEVE CERTAIN DESIRED DO LEVELS AT PINE BROOK

Parameters	Discharge	Desired DO level-ppm	Shaft H.P.size	R_s #	Number of Aerators to Achieve DO level
T = 26°C	Q = 370 c.f.s	5	50	1	8
C_o = 1.6 +Δc				2	5
C_s = 8.0			75	1	5
K_a = 0.168				2	4
L_o = 7.25		4	50	1	5
K_c = 0.2				2	4
P-R = 0.48			75	1	3
				2	3
T = 26°C	Q = 120	5	50	1	4
C_o = 1.6 +Δc				2	2
C_s = 8.0			75	1	2
K_a = 0.184				2	2
L_o = 10.5		4	50	1	2
K_r = 0.2				2	2
P-R = 0.74			75	1	2
				2	1
T = 26°C	Q = 93	5	50	1	3
C_o = 1.6 +Δc				2	2
C_s = 8.0			75	1	2
K_a = 0.201				2	2
L_o = 11.5		4	50	1	2
K_r = 0.2				2	2
P-R = 0.81			75	1	1
				2	1
T = 26°C	Q = 74	5	50	1	3
C_o = 1.6 +Δc				2	2
C_s = 8.0			75	1	2
K_a = 0.218				2	1
L_o = 12.5		4	50	1	2
K_r = 0.2				2	1
P-R = 0.90			75	1	1
				2	1
T = 26°C	Q = 54	5	50	1	2
C_o = 1.6 +Δc				2	2
C_s = 8.0			75	1	1
K_a = 0.26				2	1
L_o = 14.5		4	50	1	1
K_r = 0.2				2	1
P-R = 1.08			75	1	1
				2	1

significant result which comes directly from the simulation analysis. Table 6-1 shows results for flow rates ranging from 370 cfs to 54 cfs. The number of aerators increases with flow because both K_a and $\overline{P-R}$ decrease with increasing flow. In other words, as flow increases the rate in mg/l at which oxygen is utilized by biological reactions (i.e. $L_oK_r\exp[-K_r\tau]$) decreases less than K_a and $\overline{P-R}$ decrease. The combined effect is that there is a net increase in oxygen demand with increasing flow. In view of this demonstration, only the highest of the range flow rates (12% summer exceedance) were used in the rest of the simulation runs.

The next run was to determine the number of aerators required in the Pine Brook section to maintain a desired d.o. level of 4 or 5 mg/l, once the desired level had been achieved. This run is based on the R_s#1 method, and results are shown in Table 6-2 and Figure 6-6. The stream is assumed to enter the Pine Brook reach at the desired d.o. level. The BOD level, however, is such that if induced aeration were not present, a sag situation would develop at the downstream points. In order to cope with this situation, the locations and number of aeration stations required to maintain the desired level must therefore be determined. As in Figures 6-4 and 6-5, nitrification BOD has again been neglected. It is concluded that in order to maintain a d.o. level of 5 mg/l at a flow of 370 cfs in this reach, if there were no nitrification present, two 50 hp aerators placed 5.4 miles apart would be sufficient, *or that two 75 hp aerators placed 9.2 miles apart* would accomplish the same task. In all lesser flows, only a single 75 hp aerator would be required in order to maintain either a 4 or 5 mg/l d.o. level.

Table 6-3a and Figure 6-7 correspond to the same situation as Figure 6-6 except that the effects of a potential nitrification BOD factor are examined, which represents the situation actually found downstream of the aerator. From the simulation study in Figure 6-3, we have extrapolated the flow vs. carbonaceous BOD relationship analogously for the N_o situation to obtain a value of approximately 4 mg/l at the 370 cfs flow level. The results indicate that ten 50 hp aerators placed about one mile apart initially and two miles apart in the end reaches would maintain a d.o. level of 5 mg/l. Furthermore, seven 75 hp aerators placed between one and three miles apart would maintain the d.o. level at 5 mg/l for about 12 miles. The influence of the assumption of second stage BOD (nitrification) on the number of aerators required to maintain certain d.o. levels in the Pine Brook reach is striking. It should be observed that it has been assumed that the second stage BOD loading develops only at the first aeration station in a reach after which it decays exponentially without influence from the downstream aerators. That is, whatever mechanism starts or triggers the enhanced K_N factor, it does so only once in a 12-mile reach.

In Table 6-3b and Figure 6-8 are shown the effects of simultaneously increasing L_o and N_o by 50% over the values used in Figure 6-7, to simulate the estimated increase of organic loading in the future.

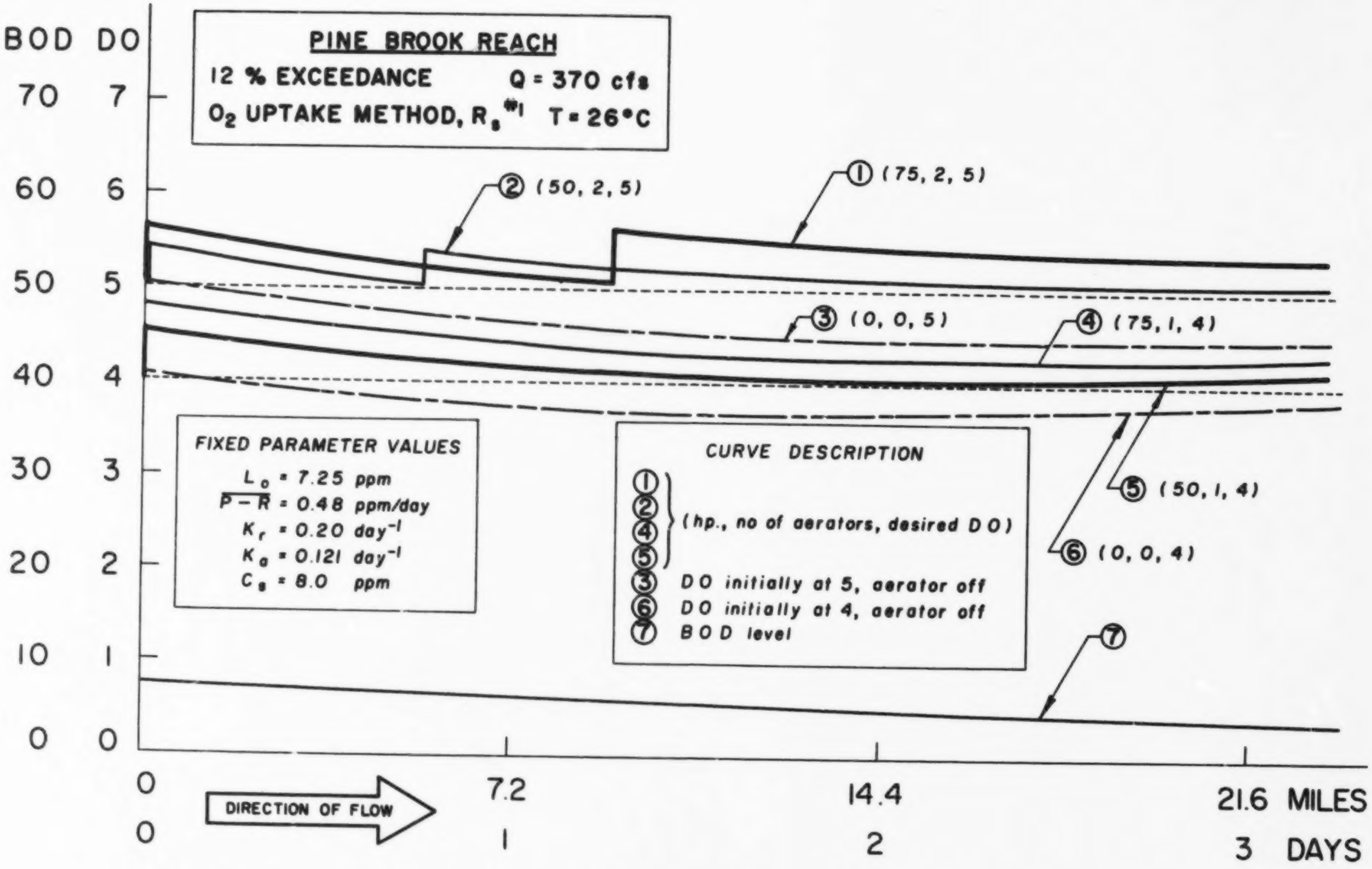

FIG. 6-6. Maintenance of DO level (relates to table 6-2)

TABLE 6 - 2

AERATOR STATION DESIGN SPECIFICATIONS REQUIRED TO MAINTAIN CERTAIN DO LEVELS AT PINE BROOK

Parameters	Discharge	Desired DO level-ppm	Shaft H.P.size	R_s #	Number of Aerators to Maintain DO level + miles from first aerator
$L_o = 7.25$ $K_r = 0.2$ $C_s = 8.0$ $P\text{-}R = 0.48$ $K_a = 0.121$ $T = 26°C$	Q = 370 c.f.s.	5	50	Method #1 used for all cases	2 - 5.4
			75		2 - 9.2
		4	50		1
			75		1
$L_o = 10.5$ $K_r = 0.2$ $C_s = 8.0$ $P\text{-}R = 0.74$ $K_a = 0.184$ $T = 26°C$	Q = 120	5	50		2 - 5.7
			75		1
		4	50		1
			75		1
$L_o = 11.5$ $K_r = 0.2$ $C_s = 8.0$ $P\text{-}R = 0.81$ $K_a = 0.201$ $T = 26°C$	Q = 93	5	50		2 - 6.1
			75		1
		4	50		1
			75		1
$L_o = 12.5$ $K_r = 0.2$ $C_s = 8.0$ $P\text{-}R = 0.90$ $K_a = 0.218$ $T = 26°C$	Q = 74	5	50		1
			75		1
		4	50		1
			75		1
$L_o = 14.5$ $K_r = 0.2$ $C_s = 8.0$ $P\text{-}R = 1.08$ $K_a = 0.260$ $T = 26°C$	Q = 54	5	50		1
			75		1
		4	50		1
			75		1

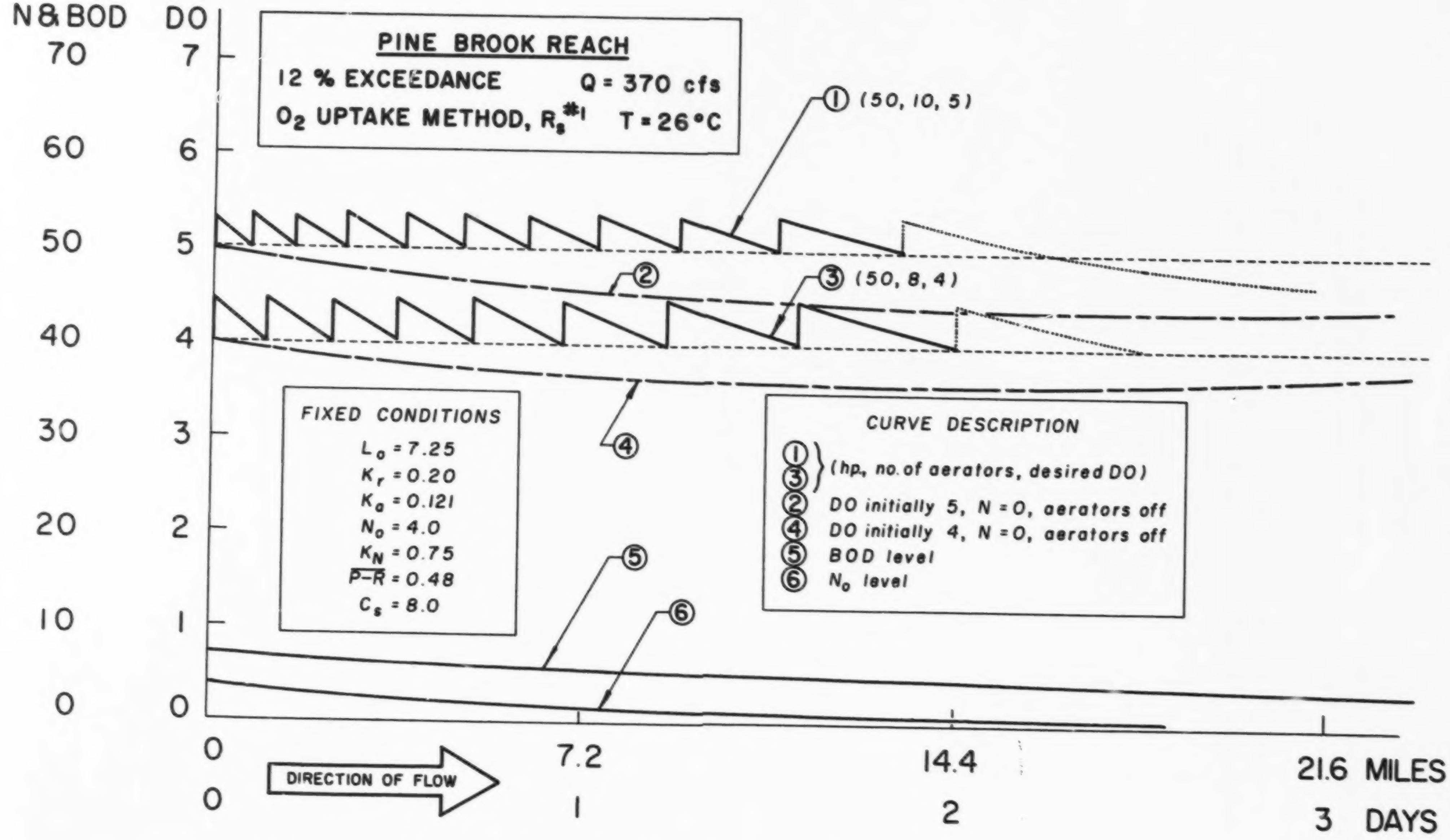

FIG. 6-7. Maintenance of DO levels with nitrification (relates to table 6-3a)

TABLE 6-3a

AERATOR STATION DESIGN SPECIFICATIONS FOR THE MAINTENANCE OF SPECIFIED DO LEVELS AT PINE BROOK WITH NITRIFICATION

Parameters	Discharge	Desired DO levels	Shaft H.P. size	R_s #	Δc	No. of aerators to maintain DO for 12 miles	Distance in miles from first aerator
L_o = 7.25	Q = 370 c.f.s.	5	50	1	0.38	10	0.8,1.6,2.6,3.8,4.9,6.1,7.5,9.1,11.0
K_a = 0.121							
K_r = 0.20			75	1	0.54	7	0.9,1.9,3.4,4.9,7.0,10.9
C_s = 8.0							
$\overline{P-R}$ = 0.48		4	50	1	0.48	8	1.1,2.3,3.6,5.1,6.8,8.9,11.4
N_o = 4.0							
K_n = 0.75			75	1	0.72	6	1.6,3.4,5.4,7.6,10.4
T = 26°C							

TABLE 6-3b

AERATOR STATION DESIGN SPECIFICATIONS FOR THE MAINTENANCE OF SPECIFIED DO LEVELS AT PINE BROOK WITH 50% INCREASE IN BOD AND NITRIFICATION

Parameters	Discharge	Desired DO levels	Shaft H.P. size	R_s #	Δc	Number of aerators	Distance in miles from first aerator
L_o = 10.9	Q= 370 c.f.s	5	50	1	0.37	17	0.5,1.0,1.5,2.1,2.6,3.2,3.8,4.4,5.1,5.8,6.5,7.3,8.2,9.2,10.4,11.5
N_o = 6							
K_a = 0.121							
K_r = 0.20			75	1	0.54	12	0.7,1.4,2.1,2.9,3.7,4.6,5.6,6.6,7.7,9.1,10.5,12.3
K_n = 0.75							
T = 26°C		4	50	1	0.48	12	0.7,1.3,2.2,3.0,3.9,4.9,6.0,7.2,8.5,10.1,11.9
C_s = 8.0							
$\overline{P-R}$ = 0.48			75	1	0.72	8	1.0,2.2,3.4,4.8,6.6,8.8,11.5

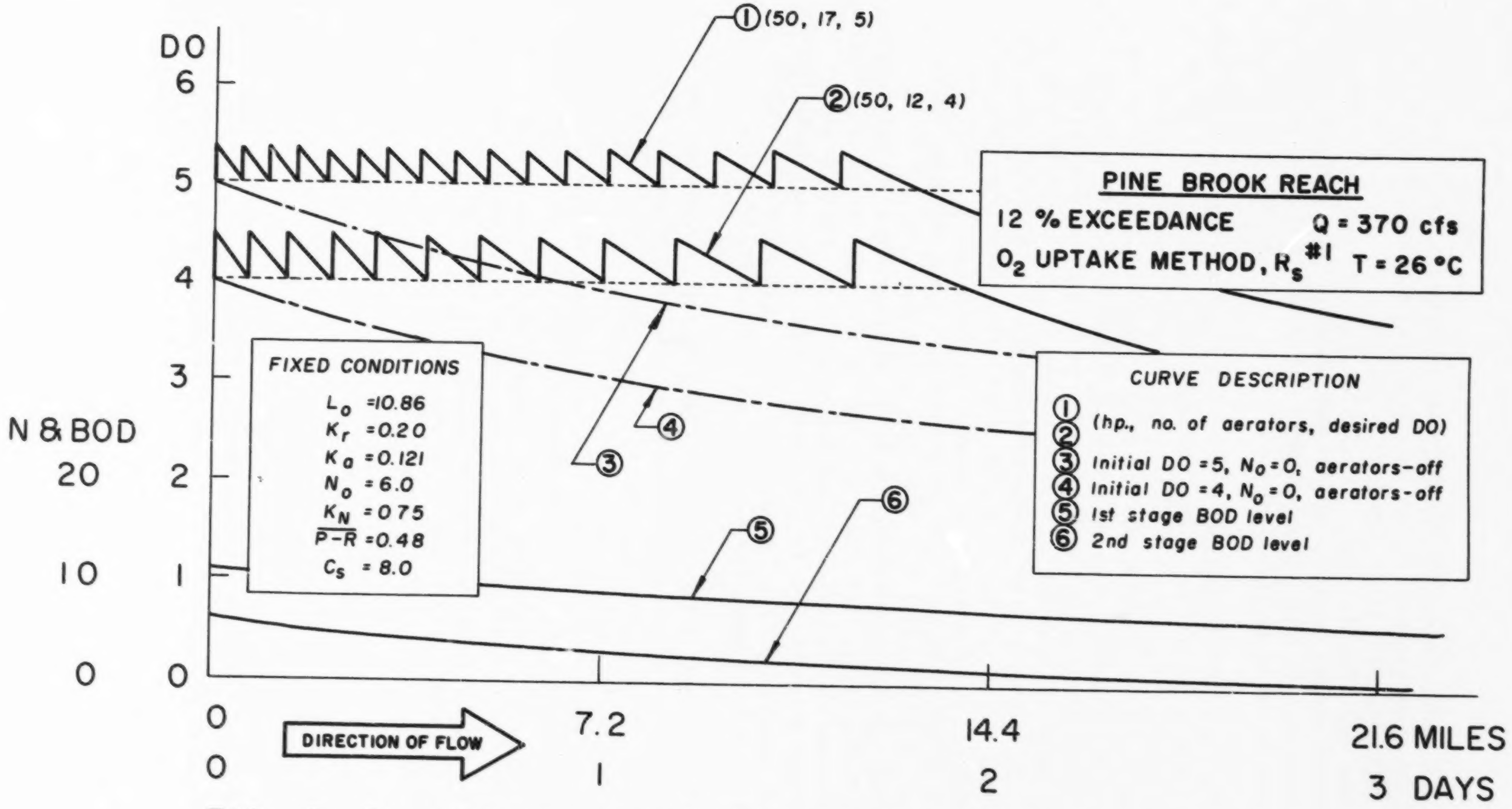

FIG. 6 - 8. Maintenance of DO levels for 12 miles with 50% increase in BOD and N_o. (Relates to table 6 - 3b.)

When increasing BOD by 50%, the number of aerators increases by 70% for the case where the d.o. level desired is 5 mg/1 and 50 hp aerators are used. However, if the d.o. level desired is 4 mg/1 the per cent increase parallels the per cent increase in BOD. In other words, increasing the BOD load by 50% increases the number of aerators by 50% at the 4 mgle desired level, but the number increases by more than 50% at the 5 mgle d.o. level.

Table 6-3c and Figure 6-9, using typical low summer flows, show an anomalous result. If the aerators are used, then according to the design figures, 8 aerators at the 5 mgle d.o. level using 50 hp aerators would be required. However, with parameters as assumed, aeration at this point would apparently not be required at all. Consequently, in this case, the potential nitrification would be passed on down the river unchanged to cause trouble in a lower reach. Therefore, by controlled triggering of nitrification by an aerator, it would be possible to utilize the river as a biological reactor for the removal (i.e. treatment) of a source of oxygen demand which is not easily treatable in an in-plant facility.

In Table 6-3d and Figure 6-10, similar results are obtained, with one exception. In this instance, because of the 50% excess BOD loading, the background d.o. sag level is such that aeration would be required in any event. The number of aerators required to maintain a d.o. level of 4 or 5 mgle using 50 and 75 hp aerators would be significant even without nitrification.

In Figures 6-11 and 6-12 a case study is made at the Little Falls section using discharges of 670 cfs, which is exceeded 12% of the time during summer. See Appendix I-1. It is observed (Figure 6-11) that at a specified d.o. level of 5 mg/1 using 75 hp aerators, 7 aerators are required to maintain the d.o. objective, whereas with a 50% increase in both BOD and nitrification (Figure 6-12) the number of aerators required increase to 11. At the lower flow (see Figures 6-9 and 6-10), the corresponding numbers of aerators are 6 and 8 respectively. The effect of using the higher flow is significant only for the situation where BOD and nitrification loads are increased by 50%.

TABLE 6-3c

AERATOR STATION DESIGN SPECIFICATION FOR THE MAINTENANCE OF SPECIFIED DO LEVELS AT LITTLE FALLS WITH NITRIFICATION

Parameters	Flow	Desired DO level	Shaft H.P. size	R_s #	Δ C	Number of aerators	Distance in miles between aerators
$L_o = 5.2$ $N_o = 4.0$ $K_a = 0.12$ $K_r = 0.2$ $K_N = 0.75$ $C_s = 8.0$ $T = 26^\circ C$ $\overline{P-R} = 0.47$	Q = 380 c.f.s	5	50	1	0.38	8	0.9, 1.8, 2.9, 4.2, 6.0, 8.0, 10.4
			75	1	0.54	6	1.4, 3.3, 5.4, 8.3
		4	50	1	0.48	7	1.3, 2.7, 4.1, 6.0, 8.5
			75	1	0.72	4	1.8, 4.5, 8.8

TABLE 6-3d

AERATOR STATION DESIGN SPECIFICATION FOR THE MAINTENANCE OF SPECIFIED DO LEVELS AT LITTLE FALLS WITH 50% INCREASE IN BOD AND NITRIFICATION

Parameters	Discharge	Desired DO level	Shaft H.P. size	R_s #	Δ C	Number of aerators	Distance in miles from first aerator
$L_o = 7.8$ $N_o = 6.0$ $K_a = 0.12$ $K_r = 0.2$ $K_N = 0.75$ $\overline{P-R} = 0.47$ $T = 26^\circ C$ $C_x = 8.0$	Q = 380 c.f.s	5	50	1	0.37	14	0.5, 1.2, 1.8, 2.4, 3.1, 3.7, 4.4, 5.2, 6.1, 6.9, 7.9, 9.2, 10.5, 12.0
			75	1	0.54	8	0.9, 1.8, 2.9, 4.0, 5.2, 6.7, 8.4, 10.4
		4	50	1	0.48	--	--
			75	1	0.72	6	1.2, 2.5, 4.2, 6.1, 8.6

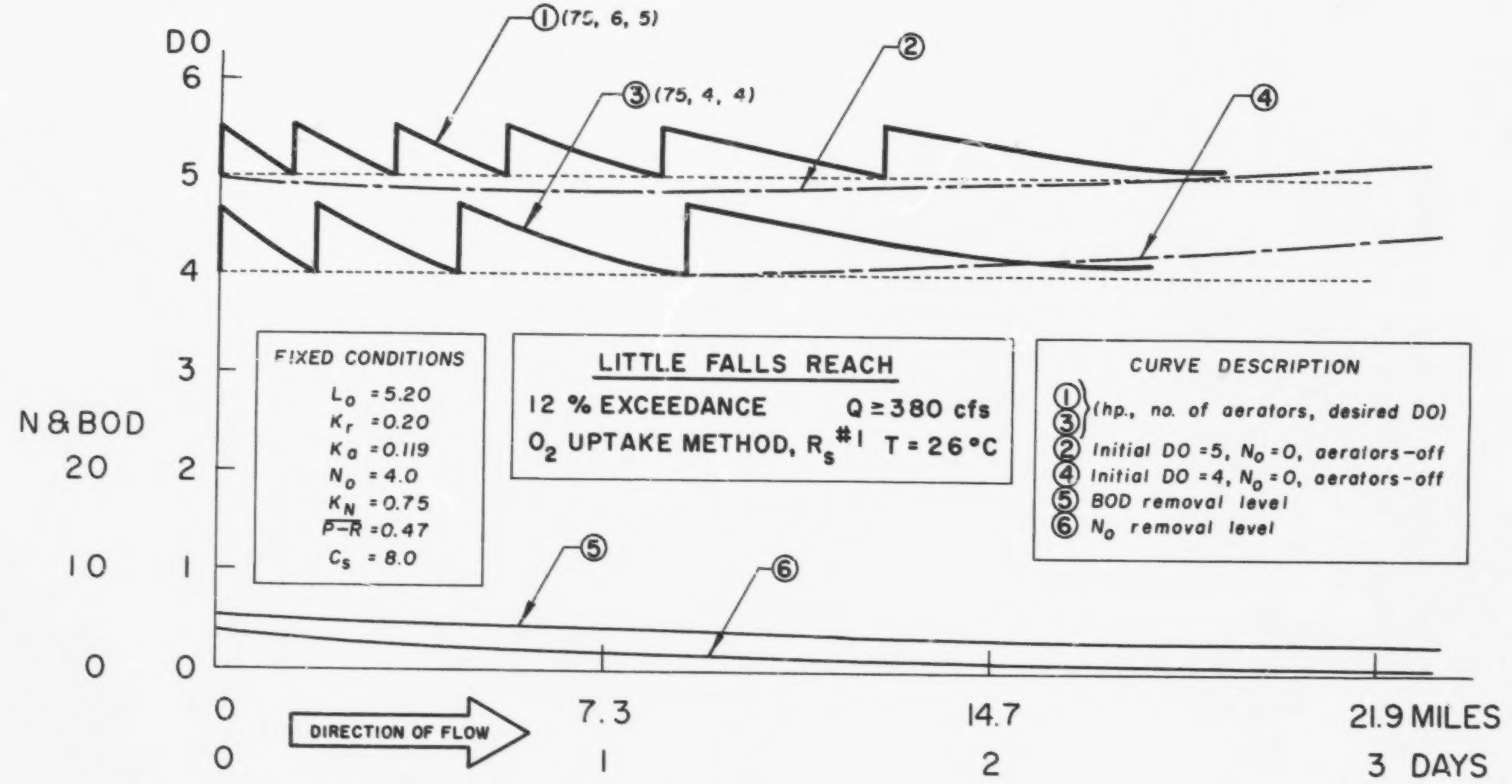

FIG. 6-9. Maintenance of DO levels with nitrification. (Relates to table 6-3c.)

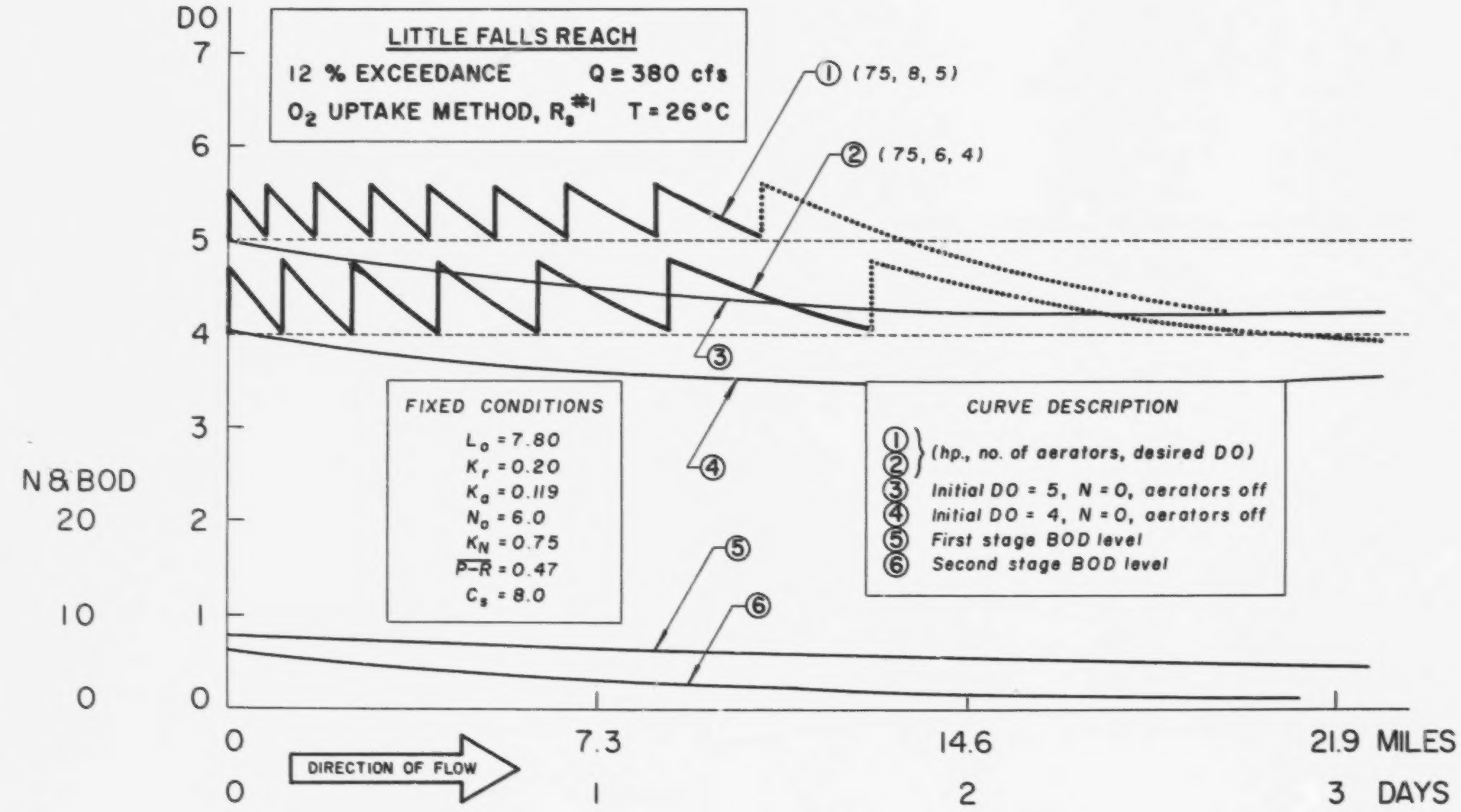

FIG. 6-10. Maintenance of DO levels with 50% increase BOD & N (relates to table 6-3d)

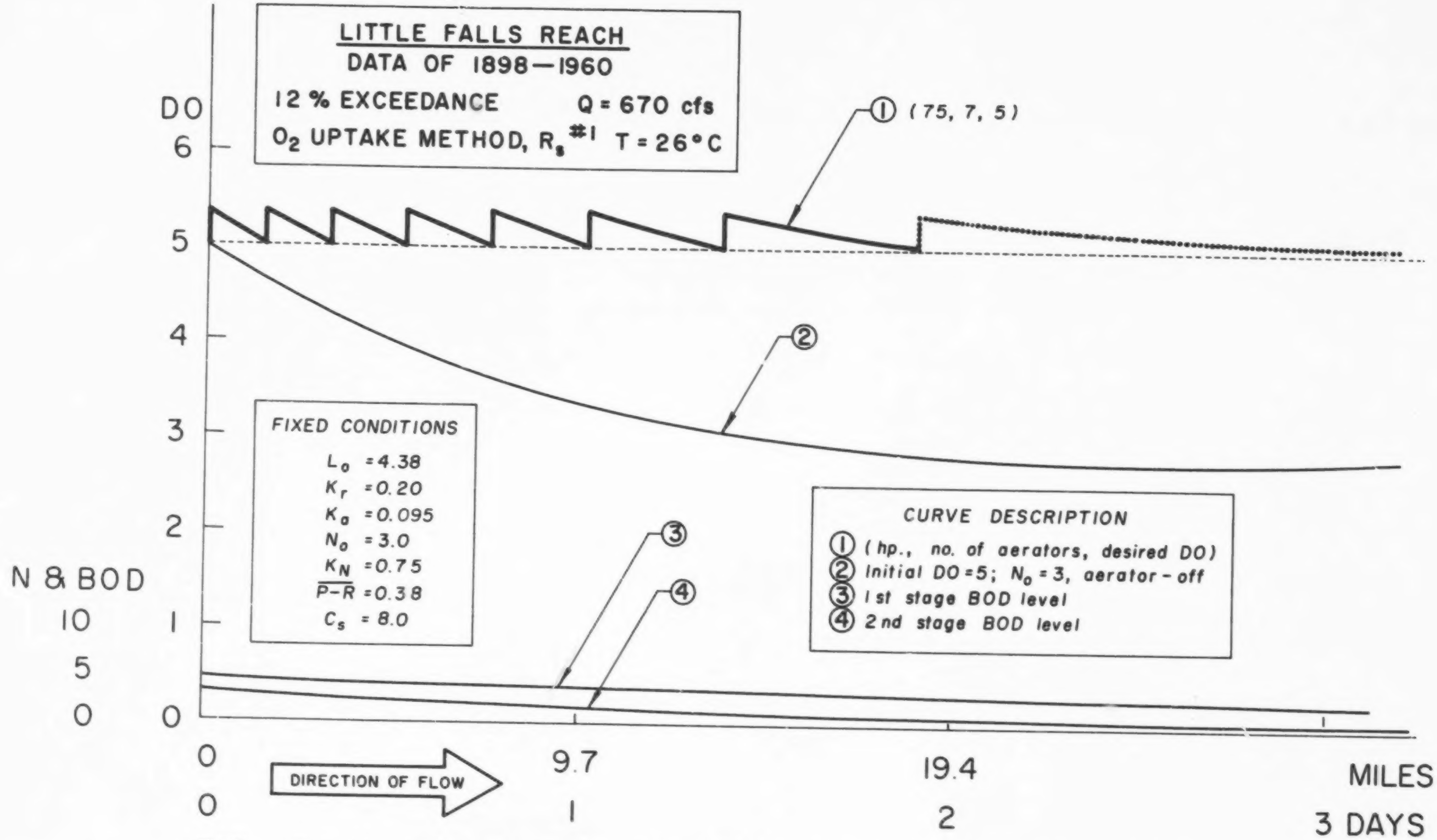

FIG. 6-11. Maintenance of DO level for Little Falls reach.
(Relates to table 6-e.)

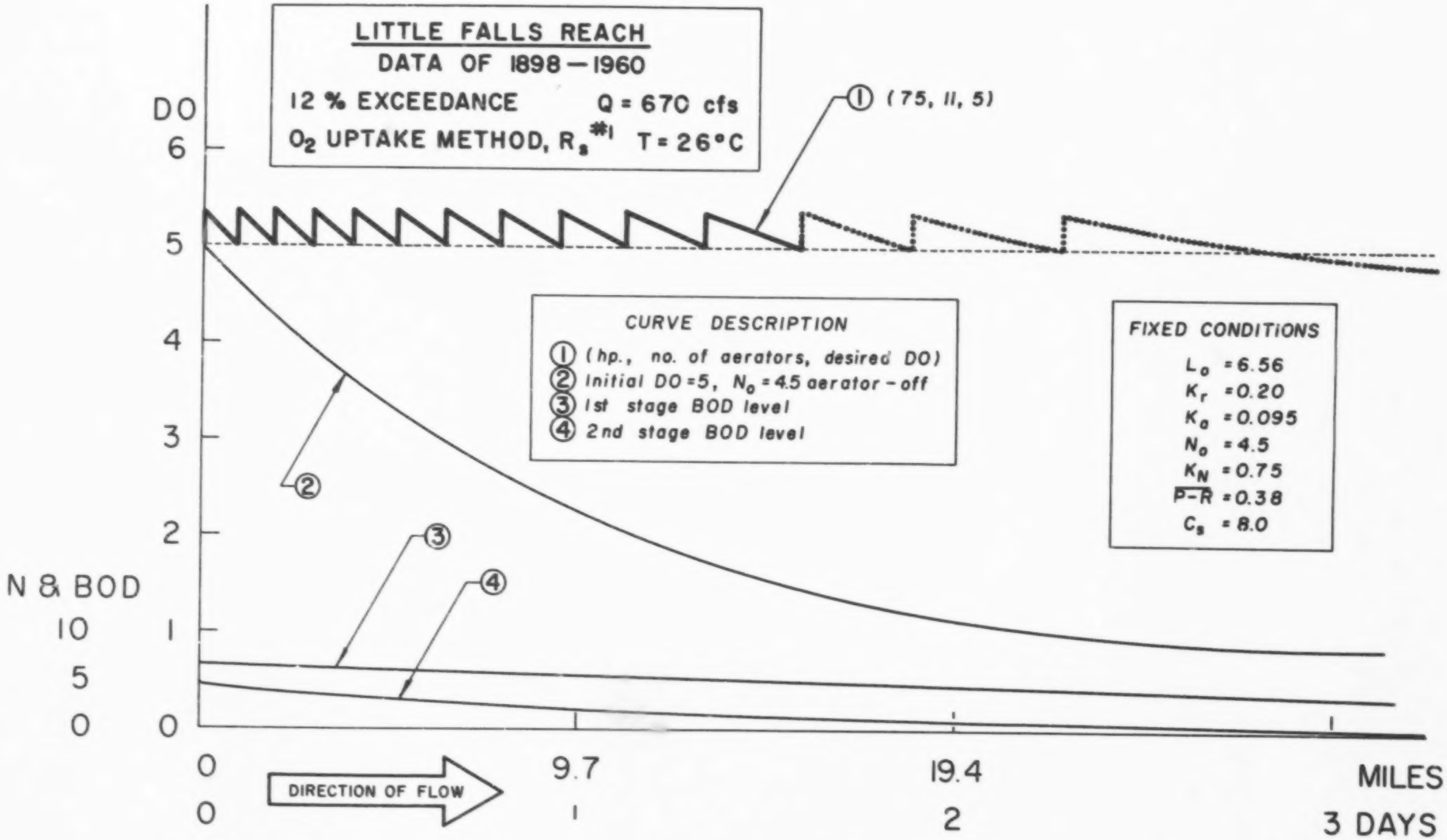

FIG. 6-12. Maintenance of DO level for 12 miles beyond Little Falls reach with 50% increase in 1st and 2nd stage BOD.

TABLE 6-3e

AERATOR STATION DESIGN SPECIFICATIONS FOR THE MAINTENANCE OF SPECIFIED DO LEVELS AT LITTLE FALLS WITH NITRIFICATION

Parameters	Discharge	Desired DO level	Shaft H.P.size	R_s #	ΔC	Number of aerators	Distance in miles between aerators
L_o 4.38	Q 670 cfs	5	75	1	0.362	7	1.5 to 3.3 avg. ~ 2.4
N_o 3.0							
K_a 0.095							
K_r 0.2							
K_N 0.75							
$\overline{P\text{-}R}$ 0.38							
T 26°C							
C_s 8.0							
$C_o = 5+\Delta C$							
$\bar{u} = 9.74$							

TABLE 6-3f

AERATOR STATION DESIGN SPECIFICATIONS FOR THE MAINTENANCE OF SPECIFIED DO LEVELS AT LITTLE FALLS WITH 50% INCREASE IN BOD AND NITRIFICATION

Parameters	Discharge	Desired DO level	Shaft H.P.size	R_s #	ΔC	Number of Aerators	Distance in miles between aerators
L_o 6.56	Q 670 cfs	5	75	1	0.362	11	0.9 to 2.0 avg. ~ 1.5
N_o 4.50							
K_a 0.095							
K_r 0.2							
K_N 0.75							
$\overline{P\text{-}R}$ 0.38							
T 26°C							
C_s 8.0							
$C_o = 5+\Delta C$							
$\bar{u} = 9.74$							

REFERENCES

(1) Dobbins, W. E., J. San. Eng'g Div., Vol. 90, No. SA 3, Part 1, pp. 53-78, June, 1964

(2) O'Connor, D. J., "Stream and Estuarine Analysis," Manhattan College Summer Institute in Water Pollution Control, New York, N.Y., Chapter 3, June, 1966.

The assistance of Mr. Paul Schorr for the computational and analog simulation work in this section of the report is acknowledged.

Chapter VII

BAISIN-WIDE ANALYSIS AND ECONOMICS OF INSTREAM AERATION

Application of Dynamic Modelling Results

When an attempt is made to apply the results of the previous chapter to the complete river system, certain uncertainties arise. In particular, the effect to be created by second stage BOD in an artificially aerated river is quite unclear; as the increased deoxygenation rate which has been shown to occur will obviously have a cumulative effect as successive reaches are improved. Table 6-3 shows that, under conditions assumed, the maintenance of a d.o. level of 4 mg/l would require six 75 hp aerators for the 12 miles reach Pine Brook - Two Bridges; and a similar analysis would presumably show that the river upstream of that point to the start of polluted conditions* would require an equal number. Maintenance of 5 mg/l of d.o. on 12 miles of river below Little Falls would require seven aerators; and although the number to maintain 4 mg/l was not computed, a proportioning to other comparisons of Table 6-3 indicates that it would probably require six. This would indicate a total of eighteen aerators of 75 hp each to maintain the desired d.o. level as far downstream as mile 17 above the estuary. The river below that point is materially influenced by major pollution from the Saddle River, coming in at mile 15.6, as reflected on the usual d.o. profile, but the analysis of Chapter VI is not sufficient to indicate what effect aeration on the three reaches upstream might have on the BOD level of this lowest reach.

Basin-Wide Model of BOD Loading

The aerator spacing indicated in Chapter VI is based upon conditions of accelerated oxygenation which were found to be created by the aerators. It is not clear exactly what causes this phenomenon. However, whatever the mechanism involved, this acceleration will obviously reduce the BOD loading which otherwise would arrive at the upper end of the next reach downstream. More specifically, any triggering of the nitrification process by the aerators would reduce the second stage oxygen demand which without the aerators undoubtedly occurs in the lower reaches of the river. As an approach to solve this problem, it is necessary to make a basin-wide accounting of BOD loading from the diverse points of origin to the lower river. This analysis must evaluate the reduction in natural aeration resulting from the higher levels of dissolved oxygen which it is desired to attain, and then consider the effectiveness, first, of induced river aeration and, second, of improved waste treatment, as a means of making up this deficiency.

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* At Chatham on the Passaic River, about 6 miles up the Whippany River, and below Boonton Reservoir on the Rockaway River, respectively.

Principle

An approximate approach would be possible using the formula

$$Y_t = L_o(1-e^{-Kt}) \qquad \text{.... Eq. 7-1*}$$

L_o = ultimate BOD at time zero,

Y_t = the oxygen used at time t,

K = a coefficient of BOD removal (assumed constant) and

t = time elapsed since time zero

If we assume, as explained in Chapter III, that BOD is removed only by deoxygenation, the K in the above equation may be interpreted as K_d, the coefficient of deoxygenation.

The conditions which are critical for oxygen level are periods of relatively high flows in the summer months, as shown in Chapter VI. Without aerators, BOD is utilized by natural aeration with oxygen levels and deficits which are somewhat different in the different reaches of the river. If these deficits, which constitute the driving force, were diminished by any given ratio by an induced aeration program which maintained higher d.o. levels, the quantity of BOD in effluents which could be handled by natural aeration would be reduced proportionately, since at every point in the stream the natural aeration would be reduced proportionately to the driving force.

However, natural aeration from the surface does not constitute the only source of oxygen for using up BOD. Some oxygen is also provided by photosynthesis, less respiration and less benthal demands. These processes cause a difference between the effective rate of BOD removal and the amount of removal attributable to aeration from the surface; and it is only the latter which can be assumed to vary proportionately to the mean oxygen deficit.

---

* Ref. Chapter III and Equation 3-20 for derivation.

For evaluation of this relationship, we may start with a different equation (see Chapter VI).

$$\frac{dC}{dt} = K_a(C_s - C) - K_rL + \overline{P-R} - B \qquad \text{.... Eq. 7-2}$$

where

C = actual d.o. level

t = time

C_s = d.o. level at saturation

K_a = coefficient of aeration

K_r = coefficient of BOD removal

L = ultimate BOD

P = photosynthesis

R = respiration

B = benthal action

Of course, there are various differences in dissolved oxygen level in different parts of the river, reflecting varying effects of biochemical oxygen demand, additional pollution loads, dilution by tributary inflow, etc. However, as an approximation, it may be assumed that the existing condition is replaced by an equivalent regimen, with a constant dissolved oxygen level and an organic load distributed proportionately to changes in natural aeration, net photosynthesis and benthal action. With these conditions dC/dt is zero: and the equation reduces to

$$K_rL = K_a(C_s - C) + \overline{P-R} - B \qquad \text{.... Eq. 7-3}$$

If the prevalent level of d.o. levels C is raised to C' by changed conditions, a reduced organic load L' will be used up by natural oxygenation as shown by the following:

$$K_rL' = K_a(C_s - C') + \overline{P-R} - B \qquad \text{.... Eq. 7-4}$$

Combining Equations 7-3 and 7-4,

$$\frac{L'}{L} = \frac{(C_S - C')K_a + \overline{P-R} - B}{(C_S - C)K_a + \overline{P-R} - B} \qquad \text{.... Eq. 7-5}$$

This relationship indicates the difference in the BOD loading at source which can be used up by natural aeration in the same stream system with different levels of dissolved oxygen.

Parameters

To evaluate parameters in Eq. 7-5, data can be obtained from Chapter III and IV as follows. The mean value of C_s is 8.3, based upon the mean temperature, 25°C, of observations taken. At the design flow of 370 cfs, K_a (20°C) for Pine Brook is approximately 0.12, from Fig. 3-2. When adjusted for 25°C by Eq. 3-18, this value becomes .152 mg/l/day per unit of driving force. Net mean photosynthetic oxygen, P-R, may be obtained from Table 3-8 for a flow of 370 cfs as .485 mg/l/day. Benthal oxygen demand for 370 cfs may be interpolated from values of Table 3-13 as .205 mg/l/day.

At summer flows of the Passaic River, which constitute the critical condition, the d.o. level of the critical Pine Brook reach is about 1.5 mg/l; but the river generally is somewhat higher, probably averaging 2.5 mg/l. If the river is to be raised to 4.0 mg/l as a minimum,* it will necessarily be raised to a higher level at aeration stations, and will diminish below each station in a sag curve which is concave upwards. The mean level will necessarily be materially above 4.0 for this reason, perhaps 4.5. (The required mean level would undoubtedly be at least as high if obtained by treatment only).

Substituting these data in Eq. 7-5 we obtain:

$$\frac{L'}{L} = \frac{(8.3-4.5).152 + (.485-.205)}{(8.3-2.5).152 + (.485-.205)}$$

$$\frac{L'}{L} = 0.74$$

This means that with the raising of the prevailing d.o., the natural aeration, aided by secondary effects, would be able to use up about 74% of the total BOD load which it now uses up.

Basin-Wide Organic Loading

The mean ultimate BOD loading introduced into the Passaic River above Little Falls has been estimated to be 86,090 lbs. daily in 1967, with a mean value of K_r of .218, based upon BOD_5 observations at Twin Falls, compared to other points above.(1) This corresponds to a K_r for 20°C of 0.293. On a year-round basis, K_r is assumed to equal K_d. Somewhat higher K_d values were indicated by laboratory tests**. However, the figure of 86,090 lbs. is already so high when compared to a total recorded effluent discharge of 27,500 (1) that it is accepted as a basis for the computations which follow, with the exception of a nitrification component which is discussed below.

---

* Values discussed here reflect daily mean conditions for the design flows, neglecting diurnal and irregular variations and conditions of higher flow. which are discussed later.

** The K_d values for 20°C computed in laboratory tests in the summer of 1968 averaged 0.356, based upon BOD_4 (Table 3-9). If these values held for the river as a whole, the organic load indicated for the basin as a whole would be even larger.

Secondary BOD

At Pine Brook, during low summer flows in 1968, the 20 day second stage BOD was found to average 16.3 mg/l (Chapter III). At a flow of 200 cfs, this would amount to approximately 17,700 lbs. of BOD ultimate, second stage. The second stage effects started with a lag time averaging 3 days. Therefore, nitrification presumably forms part of the BOD_5 measurements made at Little Falls during the summer.

Such nitrification as does occur in the vicinity of Little Falls has three effects on the interpretation of BOD data. The first effect is to exaggerate the calculated basin total of ultimate BOD load, by including in terminal BOD_5 observations some secondary BOD which is not the result of a continuous first order reaction from point of origin. If L_c is a 5 day carbonaceous BOD at Little Falls, the existence of 5 day nitrogenous BOD of L_n, this effect would result in multiplying the indicated basin organic load by $\frac{L_c + L_n}{L_c}$. Secondly, any nitrification included as part of the 5 day BOD at Little Falls would reduce the calculated value of K_r, and would thus reduce the indicated value of basin organic load. Lastly of course, the ultimate nitrogenous BOD should be added to the total carbonaceous BOD to obtain the basin total BOD.

It is impossible to determine from existing data how much nitrification may be represented in the river's flow at Little Falls, and how much of this has been included in BOD_5 measurements of the past years. Such nitrogenous BOD as was included in these observations probably had only a small effect upon the value of K_r, which was determined upon an annual basis,* but is undoubtedly of some importance in the summer. During such periods, which are critical for maintenance of d.o. levels, it is assumed that there is a load of 10,000 lbs. daily of ultimate nitrogenous BOD in the basin stream loading, over and above the carbonaceous loading of 86,090 lbs. This would require that either aerators or improved waste treatment eliminate or withhold ultimate BOD amounting to 25,000 lbs. daily, the excess over the BOD which can be used up by natural aeration under conditions assumed, or accepted as a residual load.

Induced Aeration Alternative

In order to estimate the number of aeration units which would be required to use up such an organic load, design criteria can be determined from Chapter IV. From Figure 4-9 the transfer rate is 2.1 lbs/hp-hr for standard conditions, and from Table 4-5 the coefficient of reduction of 25°C and 4.5 mg/l mean level of oxygen is 2.55 giving an actual transfer rate of .823 lbs/hp-hr. This amounts to 1480 lbs/day per 75 hp unit. For the total load of 25,000 lbs. daily to be used up by aerators, this amounts to 17 units for the river. From Table 5-1, by adding costs for two systems of 9 units, minus one third of the difference between systems of 6 and of 9, we obtain an annual cost of $194,300. This number of units, together with natural aeration, would be enough to use up the total BOD entering the system, including both second stage and first stage BOD, while maintaining the specified d.o. level.

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* Since the greatest BOD loadings occur during late winter and in the spring. At such times, temperatures are low, time of travel is reduced, and nitrification is not believed to occur in 5 day BOD's taken at or above Little Falls.

Alternative of Improved Waste Treatment

If an aerator program is not adopted, but the level of 4 mg/l d.o. is to be maintained, improved waste treatment will be required to handle the total organic load entering the stream, 86,090 lbs. of ultimate BOD daily, plus 10,000 lbs. of second stage BOD. Of this total, recorded effluents contribute 27,500 lbs. (plus undoubtedly a major part of the 10,000 lbs. of second state BOD.) and the organic loading removed from effluents prior to entering streams is 101,400 lbs. of carbonaceous BOD. If treatment is to remove an additional 25,000 lbs. of BOD by means of reducing first stage BOD, it would be necessary to reduce the 27,500 lbs. of effluent BOD, (78.7% efficiency) to 2,500 lbs. (98.2% efficiency).

The cost of improving methods of treatment of 147 waste treatment plants of various types and sizes from a mean of 78.7% to 98.2% efficiency is difficult to estimate accurately. An approximation can be made by the cost of increasing standards of treatment in plants of capacity 2.5 mgd as quoted by Kneese and Bower (2). As nearly as can be measured from the graph, it would cost $19,000 annually per mgd to increase efficiency between the two levels, in plants of this size. The estimate of 27,500 lbs. daily and ultimate BOD in effluent loading above Twin Falls may be taken as equivalent to about 41.2 mgd of average domestic waste (assuming raw waste of 375 BOD ultimate and 78.7% efficiency of treatment). This volume at the cost indicated would correspond to a total additional cost of $785,000 annually for the basin above Twin Falls. The actual cost would probably materially exceed this amount, on account of the large number of treatment plants, most being of small size. However, it is possible that some means of reducing nitrogenous BOD would provide an alternative, to offset this increase.

The above estimate of $785,000 for improved waste treatment is directly comparable to the estimate of $194,300 for accomplishing the same objective of 4 mg/l through induced aeration.

Level of 5 mg/l d.o.

A similar comparison may be attempted to achieve a general minimum d.o. level of 5 mg/l rather than 4. The mean level may be taken as 5.4 mg/l.

Substituting in Eq. 7-5, it is shown that at this higher level, natural aeration would only be sufficient to use up 60.4% of the BOD now consumed in the stream. Continuing as before, it may be shown that this would require 38,000 lbs. of BOD to be used up by other means in order to meet the desired conditions. Since this amount exceeds the entire effluent loading, nitrogenous as well as carbonaceous, there is no possibility of meeting this objective through treatment only.

Turning to induced river aeration, and using the transfer rate of 2.1 lbs/hp-hr, as before, the coefficient of reduction for 25°C and 5.4 mg/l is 3.34 giving an actual transfer rate of .628, or 1130 lbs/day per 75 hp unit. For the reduction of 38,000 lbs., this amounts to 34 units of 75 hp for the river, which from Table 5-1 may be estimated to cost $388,600 annually.

The greater advantage of the induced river aeration in relationship to achieving a general level of 5 mg/l, as compared to a level of 4 mg/l relates mainly to the large amount of unrecorded pollution in this stream. Treatment plants, which control less than one third of the total organic loading in the Passaic River, would be unable to achieve a level of 5 mg/l; whereas the induced aeration stations, which act upon the entire organic load, could maintain a general level of 5 mg/l without difficulty.

It should be noted that, as between the two methods, the induced river aeration appears the more certain in its application. The aerators can be placed so as to act upon the river wherever their action will be most effective. On the other hand the treatment of effluents at site would be almost sure to leave an unbalanced total loading, such that an established average d.o. level would not in fact result in this level being maintained at all points as assumed, but would leave parts of the river in uneven sag curves below the standard.

The ratio of comparative costs to achieve a d.o. level of 4 mg/l is not quite as great as that estimated by Davis for the Potomac River (3). However, Davis assumed waste treatment raised to a general level of 90% efficiency prior to outlining of alternatives, whereas the Passaic River current waste treatment level of 78.7% was taken as a starting point.

Clearly the number of aeration stations given above for the Passaic River would not long keep the river in a satisfactory state. Informal estimates of the FWPCA have indicated that despite anticipated improvement in levels of treatment, the total effluent waste loading of the basin is expected to increase by almost one-half by the year 2010 (4). Under such conditions, an even greater number of aerators would be required, even if there were to be no increase in water supply withdrawals. With such increased effluent loadings, and a concomitant increase of unrecorded pollution, there appears to be no chance whatever of maintaining satisfactory d.o. levels by effluent treatment alone.

Reliability of Specified d.o. Levels

The above computations were made upon the basis of maintaining a given mean daily average d.o. concentration at specified design flows, with a BOD concentration which is the mean of values occurring at such flows.

Neither the river aerators nor the waste treatment alternative could maintain standards 100% of the time. for reasons which are partly common to both and partly particular to each one. Although a complete statistical analysis is beyond the scope of this study, the more important aspects are summarized in the following as paragraphs.

As regards waste treatment, the organic load of effluents is recorded as being substantially the same at different times of the year, although seasonal stream discharges are very different. BOD_5 concentration in the stream is reduced at higher discharges; for example at Pine Brook in 1967, the concentration was about 12.5 at 100 cfs and 3.5 at 1000 cfs. However, it may be seen from these data that total organic load is almost three times as great at the higher discharge. Undoubtedly the large unrecorded components of pollution occur in much greater amount at the higher discharges; and whatever degree of treatment of effluents is provided will be greatly reduced in its effect upon the total load when discharge is high.

The river aeration alternative is somewhat better adapted to this situation, since it treats the river's organic load as a whole and not only a part of it. However, the discharges on the basis of which the aeration systems were designed (370 cfs at Pine Brook and 670 cfs at Little Falls) are exceeded 12% of the time in summer, which is the critical period. See frequency curve, appendix I-1. Flows in excess of the design flow may be expected to have an organic load in excess of the design capacity; and such flows, up to more than three times the design flow, may still have undesirably low dissolved oxygen.

The second reason for levels of d.o. to fall below the established norm is stochastic variation in BOD concentrations. All calculations have been based upon average BOD levels for a given discharge; but the actual relationship is somewhat irregular. In order to obtain an evaluation of this situation, the daily determinations of BOD made by the Passaic Valley Water Commission were plotted for the three months July-Sept. 1967, as shown in Fig. 7-2. Deviations are considerable at all ranges of discharge, the standard deviation from the curve shown being about 1.6 mg/l. Moreover,

FIG. 7-1

BOD-DISCHARGE RELATIONSHIP
PASSAIC RIVER, PINE BROOK

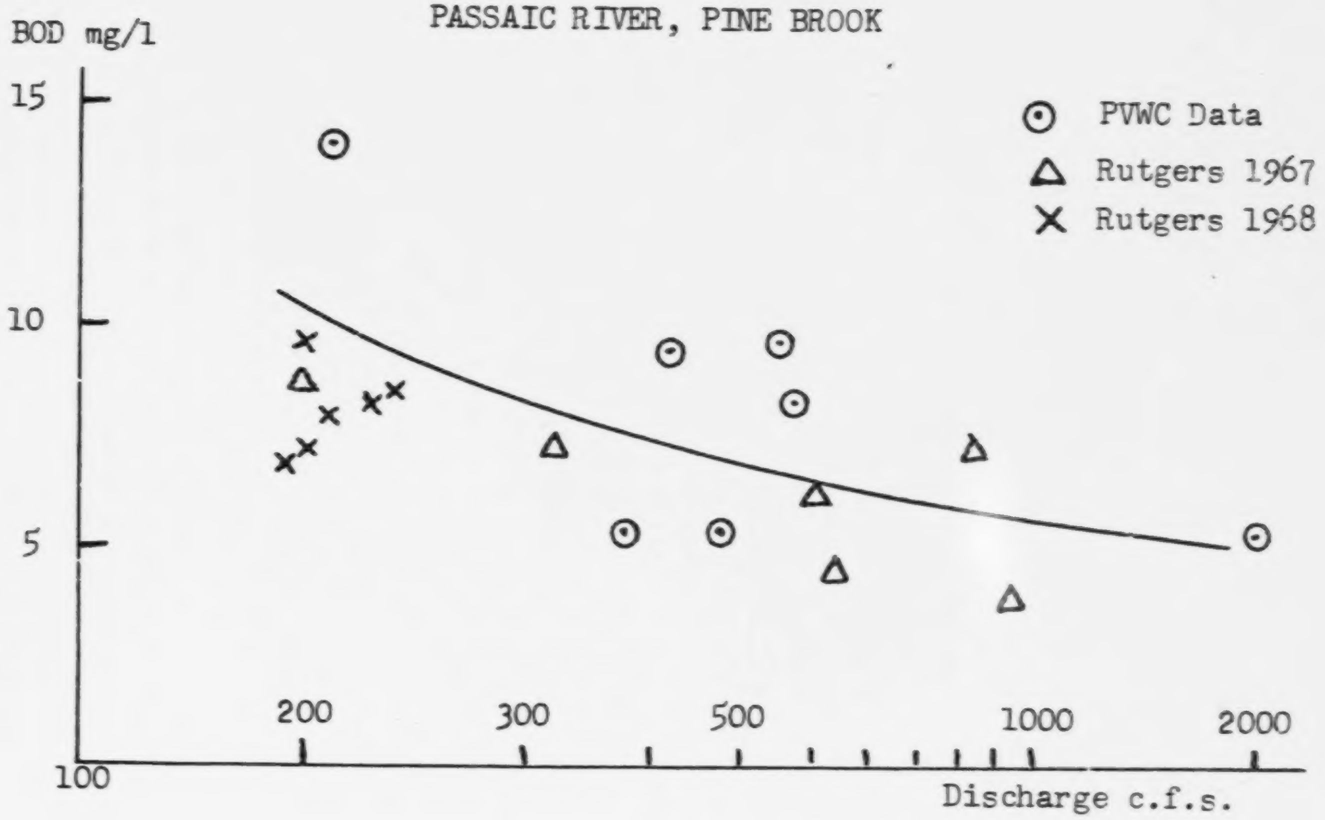
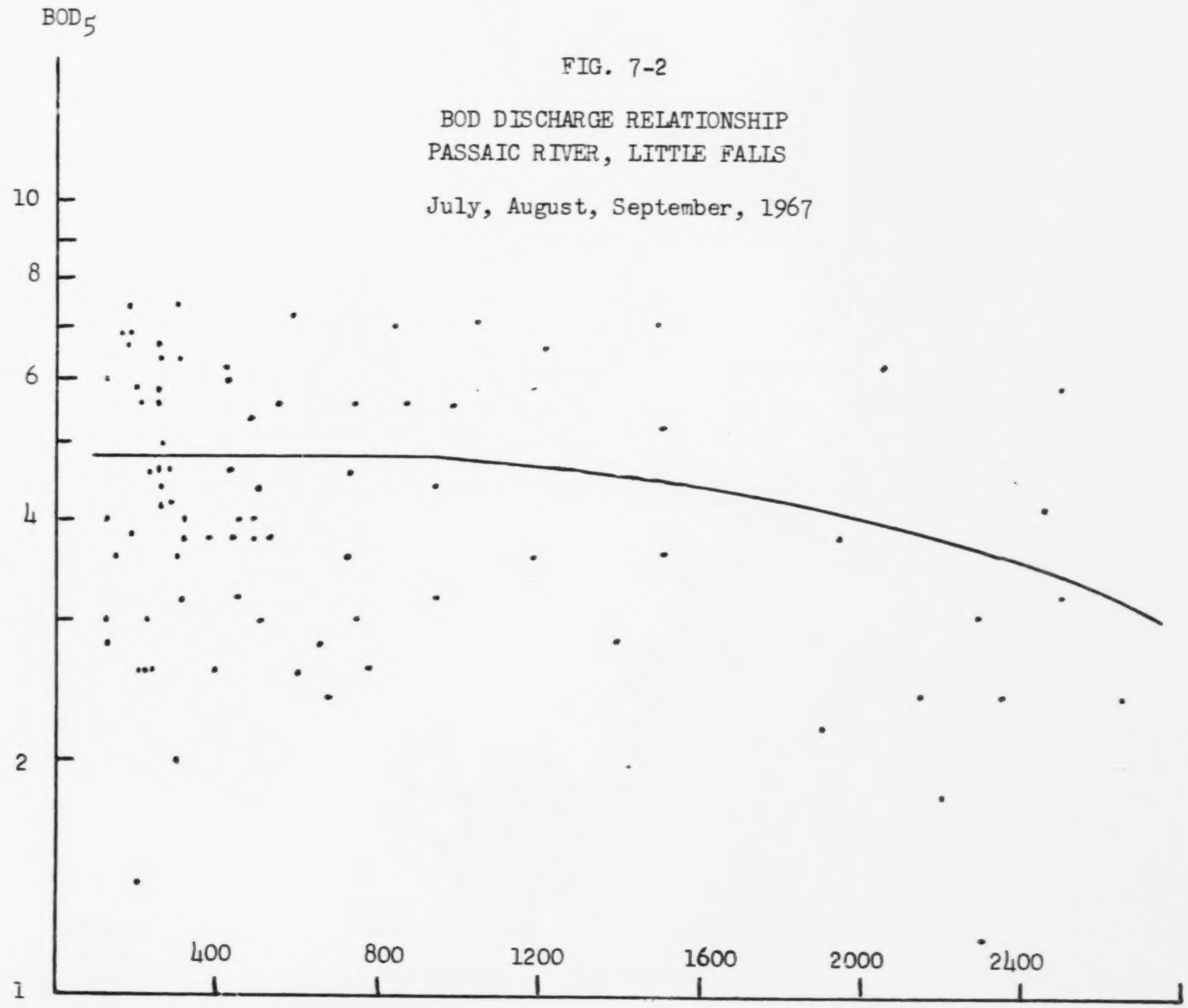


FIG. 7-2

BOD DISCHARGE RELATIONSHIP
PASSAIC RIVER, LITTLE FALLS

July, August, September, 1967



there are considerable shorter period variations. Figs 7-3 and 7-4 show observations of BOD taken at half hour intervals on a single day, respectively (1) at site #1 and (2) starting at site #1 and floating continuously downstream so as to follow the same water mass. The observations moving downstream show quite good continuity, with a downtrend of BOD consistent with the process of biochemical oxygen demand in a reach with no additional sources of pollution. There was one very high reading, which might have represented a bit of solid matter in the sample. The samples taken over the same period of time at site #1, on the other hand, show not only a generally rising trend, but much greater fluctuation, and one irregularly high period of three hours. It is apparent that there must be many occasions during which the discharge of the river is somewhat below the design level but prevailing BOD loading in the water mass in question is well above what is normal for that discharge.

It may be concluded that, of the 12% of time discharge exceeds the design level, there will be perhaps two-thirds, or 8% of time, in which total BOD will exceed the levels for which the aeration system were designed. Moreover, there will be many discharges below the design discharge at which deviations of BOD concentration above the normal will produce total loads which exceed the design conditions. No formal statistical analysis has been made; but it appears likely that the design level of 4 or 5 mg/l as specified will fail to be maintained perhaps 15% of the time for these reasons.

In addition, there is another reason for levels to fall temporarily below the design level, namely photosynthesis and algae respiration. Diurnal fluctuations of d.o. occur during the summer, and commonly amount to 0.5 mg/l or more below the daily mean for a considerable period each night.

It is apparent that a very detailed statistical study would be required to determine the true reliability of specified d.o. levels, and the percentage of time which they might be expected to be maintained. Preliminary considerations given above indicate that the general levels of 4 or 5 mg/l might be held during the day time for about 85% of the time, with further temporary deviations at night. If it is desired to hold a specified d.o. level as a minimum for a greater proportion of the time, a higher general level must be achieved.

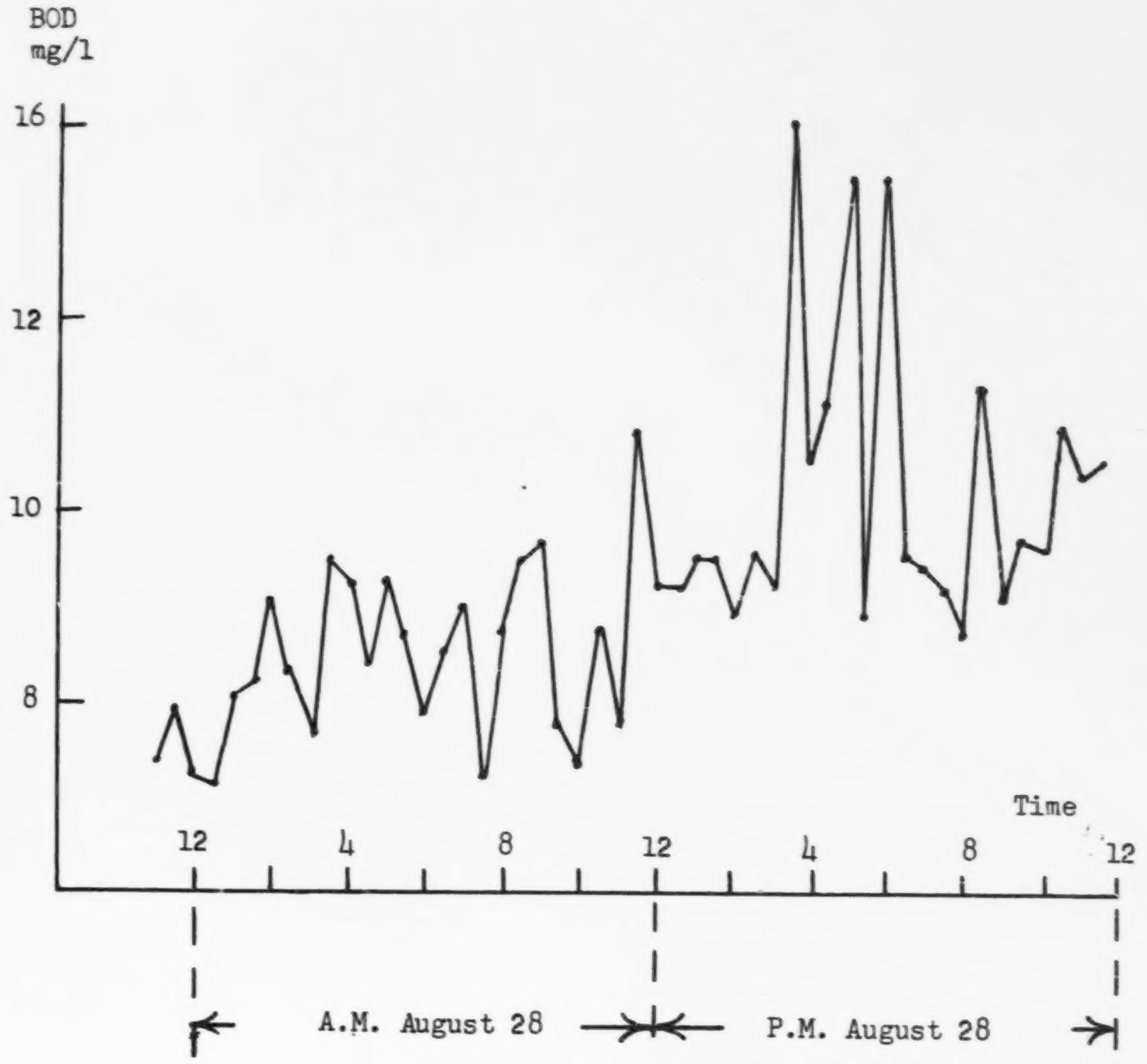


FIG. 7-3

24 HOUR BOD OBSERVATIONS

STATION -200

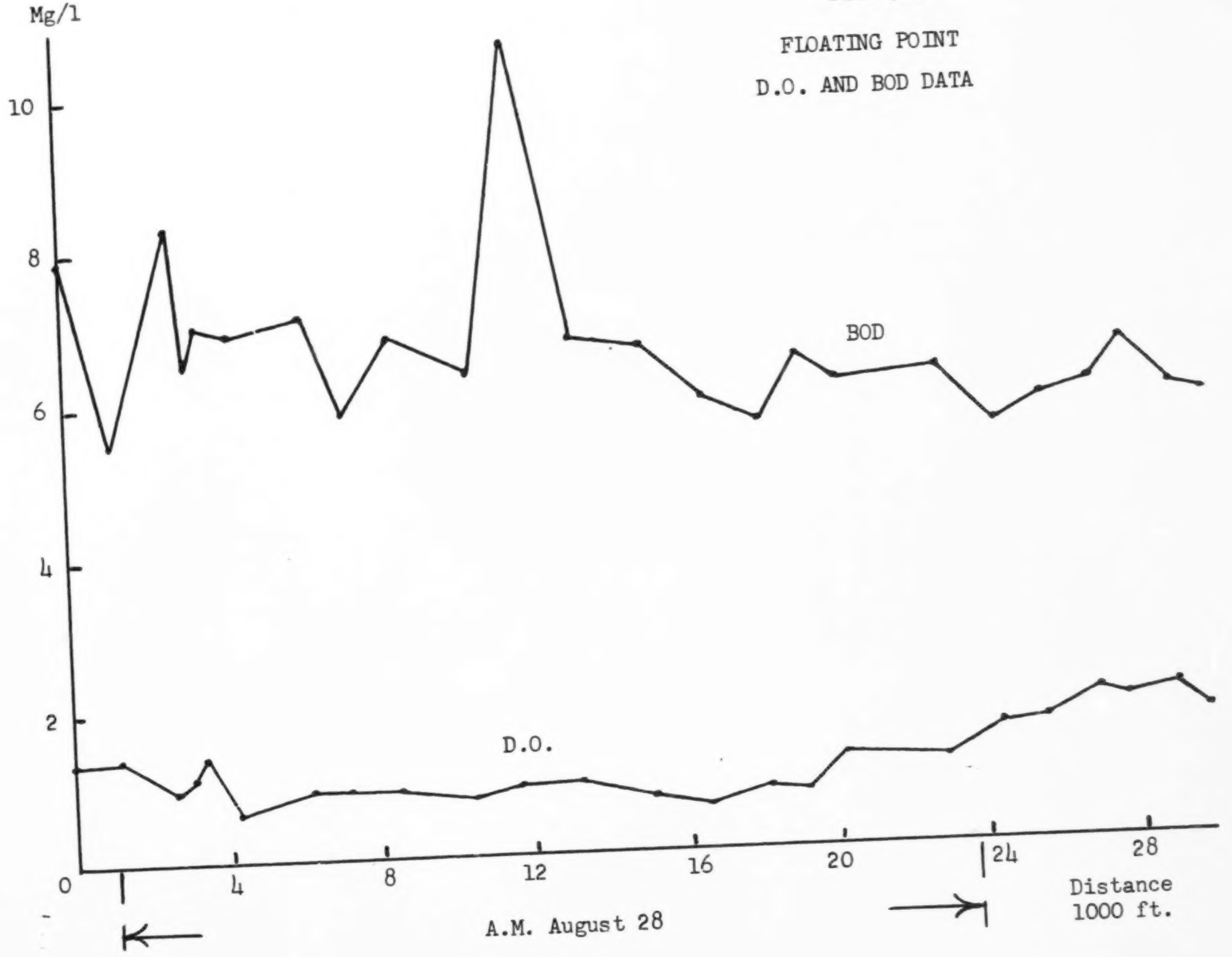


FIG. 7-4

FLOATING POINT

D.O. AND BOD DATA

REFERENCES

(1) Unpublished data from the study, Whipple, W. and Marcus, M., "Urbanization and its Effect Upon Water Resources," OWRR, B - 002 - N.J. See. Chapter I.

(2) Kneese, A.V. and Bower, B.T., "Managing Water Quality: Economics, Technology, Institutions," Johns Hopkins Press for R.F.F., 1968, p. 53. The source quoted is Frankel, R.J., "Cost Quality Relationships in an Engineering-Economic Model for Municipal Waste Disposal," ASCE Water Resources Engineering Conf., Mobile, Ala., 1965.

(3) Davis R.K., "The Range of Costs in Water Management" Johns Hopkins Press for RFF, 1968.

(4) Informal estimates of Federal Water Pollution Control Administration, Metuchen, N.J. office.

Chapter VIII

GENERAL CONCLUSIONS

1. For small polluted rivers such as the Passaic, induced river aeration is an entirely practicable alternative to advanced stages of waste treatment, or to low flow augmentation, and apt to be a much less expensive means of reaching desired water quality standards. It should be considered as supplementary to secondary treatment of effluents.

2. The methods of evaluating aerator performances in lagoons and tanks are not applicable to rivers, except with major modification.

3. Figures sometimes cited of oxygen transfer rates of 2-4 lbs./hp-hr in rivers are quite misleading if not qualified. Actual performances tested approximated 2.1 lbs./hp-hr, reduced to standard conditions, for normal summer flow rates, but at summer temperatures, when maintaining actual river water at a d.o. level above 4 mg/l, the performance to be expected would be about 0.8 lbs./hp-hr. Adjustment of transfer rates to actual conditions should be based upon values of parameters expected.

4. Diffuser type aerators were markedly less economical than mechanical aerators under conditions tested, due to lower transfer efficiency.

5. The aerators actually used functioned quite satisfactorily, but certain improvements could be made for river operation. In particular, they develop an irregular amount of power, at times well above and at times substantially below rated capacity. Some better regulator of output should be developed for river use, since methods which can be used in tanks are not applicable in rivers.

6. On the assumption of equal efficiency of 50 hp and 75 hp aerators, the optimum system for a river similar to the Passaic would be based upon 75 hp aerators. The advantage of smaller aerators in providing wider distribution of aeration sources seems to be overcome by the cost of installing and maintaining the additional number of installations.

7. The mechanical aerator was tested in a basin about 100 ft. square and 8 feet deep at low water, excavated in the bed of the river, which was originally 2-4 feet deep in most of its channel. The diffuser aerator basin was excavated a little deeper. The actual depths were used in all cost estimates; but there is no evidence to prove that somewhat better results might not be achieved with some other depth, or channel configuration.

8. The mechanical aerator shows a marked increase in transfer rate with increases in mean stream velocity from a low of about ¼ ft./sec. to a high of about 1.0 ft./sec. (in the unexcavated channel). It is considered likely that an increase in efficiency could be achieved under low flow conditions if the velocity of approach were increased by concentrating flow in the center of the channel; but this hypothesis has not been tested.

9. Use of aerators may result in a great increase in deoxygenation coefficient, amounting to several times its normal amount, and extending several miles downstream. It is tentatively assumed that this effect is caused mainly by accelerating second stage or nitrogenous BOD reaction, but it may be due in part to increase in primary BOD reaction, or to passing into suspension of benthal deposits. More research on this point is essential. If this effect upon nitrogenous BOD can be verified, it will be an additional advantage to induced river aeration.

10. Dynamic modelling furnishes an excellent guide for spacing of aerators where parameters can be evaluated. However, due to the uncertainties referred to in 9, above, a basin-wide analysis of BOD loading, from its sources to the lower parts of the stream, is essential in order to determine the total number of aerators required. Such an analysis is equally necessary to predict the results of any other kind of water quality control program.

11. On the Passaic River, and probably on many other rivers, the existence of large amounts of unrecorded man-made organic loading makes it unrealistic to attempt to attain high levels of dissolved oxygen through treatment only. Induced river aeration may be not only an economical alternative, but the only practicable way of achieving such goals.

12. This demonstration and research program has shown the engineering practicability and effectiveness of induced river aeration, and its economic feasibility as compared to other alternatives. No inference should be drawn from this report as to who should pay for such a program.

13. The very pronounced economies and advantages of induced aeration cannot be developed without a scientifically adequate analysis of the BOD loading of the entire basin, an economic comparison of alternative means of water quality control, and the creation of a technically competent agency to exercise general surveillance over waste disposal from all sources. Merely to place a set of aerators in the stream would be unlikely to produce the desired result; it might even encourage industries and towns to delay needed waste treatment improvements. Present institutional arrangements will require major modification to adapt to a river aeration system; but these arrangements require strengthening in any event, if the adopted water quality standards in rapidly developing areas are to be taken seriously.

Acknowledgement

This report was supported in part by a demonstration grant No. 16080 DUP from the Research and Training Grant Program, Federal Water Pollution Control Administration, in part by financial support from the Department of Conservation and Economic Development, State of New Jersey, and in part by several research grants from the Office of Water Resources Research. Department of the Interior.